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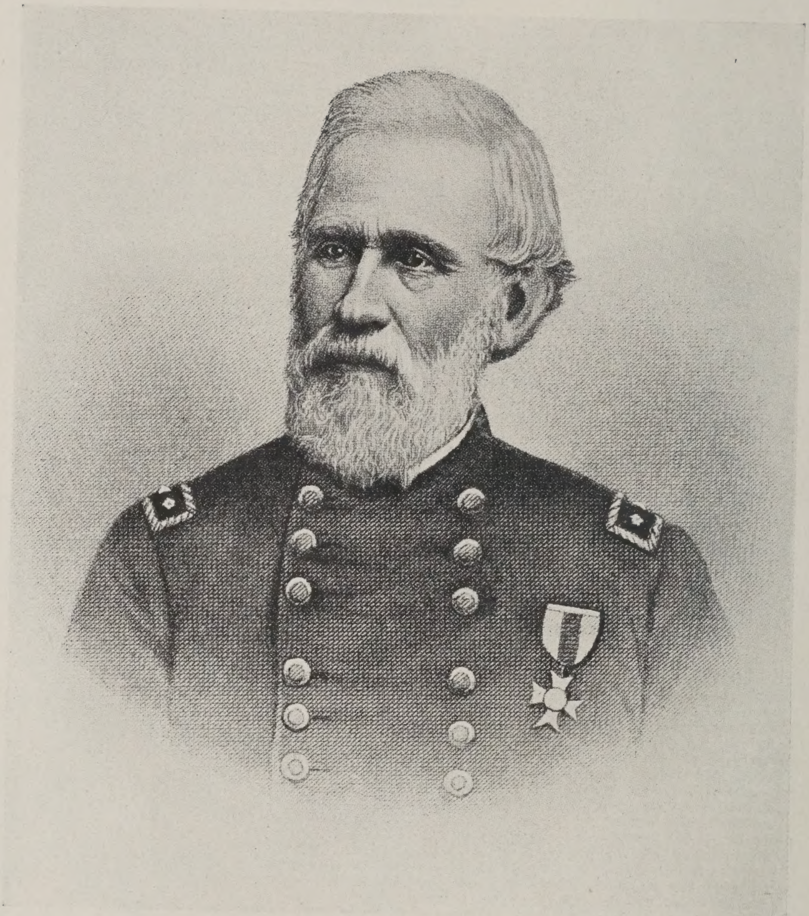

GEN. JOHN GROSS BARNARD

CORPS OF ENGINEERS, UNITED STATES ARMY

1833-1881

BORN 1815—DIED 1882

SEE P. [illegible]3

Mobile Harbor, Alabama

BY

Capt. C. O. SHERRILL
Corps of Engineers

Mobile is a city of about 52,000 people, situated in southwest Alabama at the outlet of Mobile River into Mobile Bay. It is the only seaport of the State and the most important one on the Gulf east of New Orleans, both in tonnage and value of its maritime commerce.

The country naturally tributary to Mobile is the whole Alabama River Basin, about 33,000 square miles, including the greater part of Alabama, eastern Mississippi, and western Georgia. This basin has an average rainfall of 54 inches, making a total of 28 cubic miles of rainfall annually. This volume of water is carried through the Oostenaula, Etowah, Coosa, Tallapoosa, Little Tombigbee, Black Warrior, Tombigbee, and Alabama rivers into Mobile Bay through the Mobile River Delta.

Being thus favored by nature in its location at the outlet of the largest river system in the United States, except that of the Mississippi and the Columbia, with over 1,000 miles of navigable streams behind it, Mobile should be able to secure a large share of the tonnage passing through the Gulf ports and the Panama Canal; especially with the coaling facilities which should be developed as a result of the canalization of the Warrior and Tombigbee rivers into the coal fields of the State.

Mobile has an interesting history, varied as it is into periods under five consecutive flags: Spanish, French, British, Spanish, and American. Mobile Bay was first visited by Europeans in 1500, and in 1519 the Governor of Jamaica sent an exploring party along the Gulf Coast under Pineda, who named the bay and river "Espiritu Santo." In 1558, Bazares came from Mexico to explore this territory, and in 1559, the first settlement was made by Tristan De Lima. Two Frenchmen, Iberville and his brother Bienville, explored Mobile Point and Dauphin Island, in 1696, and made al-

liances with the Choctaws, which formed the basis of the powerful French influence throughout the Mississippi Valley. Bienville, in 1702, located Mobile (named for the "Maubila" tribe of Indians) at the mouth of Dog River, but, due to an inundation by storms, the town was shifted in 1711 to its present site. In 1763 the British captured the town, then called Fort Condé, and named it Fort Charlotte, holding it for seventeen years when it was taken again by the Spanish. General Wilkinson finally captured Mobile, in 1813, and incorporated it as a part of the United States.

From 1818 to 1860 Mobile grew rapidly as a cotton port, reaching its greatest prosperity just previous to the Civil War when 800,000 bales of cotton were received. Following this war there was a steady advance until the panic, 1873, brought business to a standstill for eight or ten years, since when there has been a slow but steady growth. The building of railroads, the opening up of additional harbors on the Gulf, and the failure of the city to secure factories, have been the principal causes of the slow growth of this port.

Mobile Harbor is taken to include Mobile Bay and Mobile River for about 5 miles above its mouth. It is essential therefore, in a study of the development of this harbor, to become familiar with the characteristics of both river and bay. Mobile River is formed by the confluence of the Tombigbee and Alabama rivers, 45 miles above Mobile. From this junction the flow is limited to a single channel for 5 miles, then at successive cut-offs the river divides into a number of estuaries which, under the name of Mobile River, Spanish River, Apalachee River, Blakeley River and Tensas River, flow into the Bay through a flat, partly wooded and partly marshy delta of 250 square miles area. The more westerly one of the minor estuaries, Spanish River, always had, in its natural state, a deeper and more uniform channel than Mobile River proper; so much so that previous to Government improvement of this harbor, vessels came into Mobile by the roundabout route up Spanish River, then down Mobile River to the docks. The earliest recorded available depth in Spanish River was about 8 feet, whereas across Mobile River Bar (Choctaw Bar), the depth was scarcely 5½ feet. After a detour to the east, Spanish and Tensas rivers again approach, in a south-westerly direction, the main channel as it enters the Bay, and thus their combined flow helped to keep open the old channel from the mouth of the river to the elbow (Beacon 16). On the other hand, the waters of Mobile River passing into the Bay and

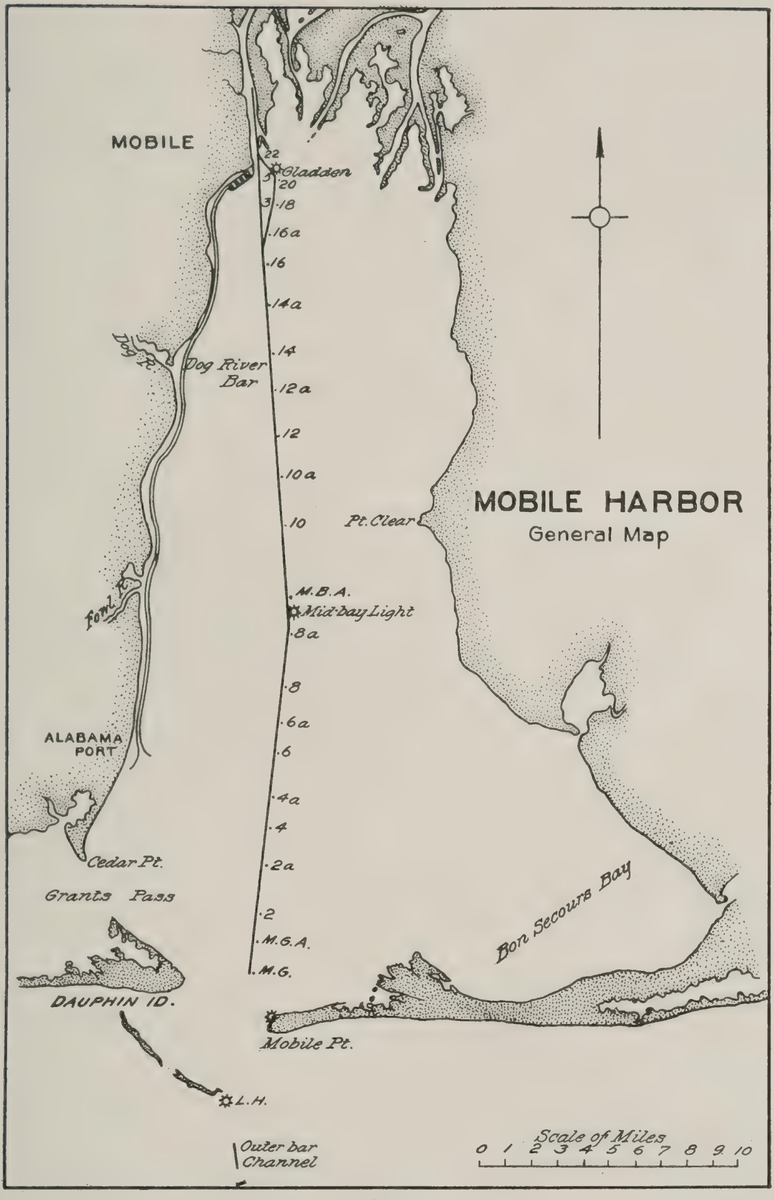
MOBILE HARBOR
General Map
MOBILE
Gladden
22
20
18
16a
16
14a
14
Dog R.
Dog River Bar
12a
12
10a
10
Pt. Clear
M.B.A.
Mid-bay Light
8a
Fowl R.
8
6a
6
ALABAMA PORT
4a
4
2a
Cedar Pt.
Grants Pass
2
M.G.A.
M.G.
Bon Secours Bay
DAUPHIN ID.
Mobile Pt.
L.H.
Outer bar Channel
Scale of Miles
0 1 2 3 4 5 6 7 8 9 10

Fig. 1.

simultaneously meeting the stronger currents of Spanish and Tensas rivers, were robbed of their velocity and converted into an eddy, which caused the formation of Choctaw Bar.

Mobile Bay is an arm of the Gulf of Mexico, having an area of 375 square miles. It extends from the outer bar 30 miles inland, with a width varying from 60 to 20 miles, and a natural depth decreasing from 57 feet opposite Mobile Point to 5 or 6 feet at Choctaw Bar. In addition to the above-mentioned rivers, there are two small streams entering the Bay on the west. These are of little importance as silt bearers. Extensive borings to a depth of 26 feet, made in 1878, from Mobile to the lower anchorage, indicated the same conditions of bottom for given depths, viz: sand mixed with mud and shells to a depth of 14 feet; below this, to 26 feet, a stiff, tenacious clay, hard on top, growing softer below. The same formation was found on the bluffs on the western shore, the clay being about 12 to 15 feet from the surface or at the water level. The surface strata in the Bay has a specific gravity of about 1.3. A line of levels made in 1878 from Cedar Point to the Mobile and Montgomery Railroad bridge indicated a fall in the water surface of 0.75 foot in 46 miles. Sediment observations gave an average of 100 grains per cubic foot of water in the rivers.

The discharge of the Bay passes into the Gulf of Mexico between Mobile Point and Dauphin Island, and into Mississippi Sound south of Cedar Point. The dredged channel extends from the 23-foot contour in the lower Bay northward along the line of deepest water to Chickasaw Creek, 5 miles above the mouth of Mobile River. This channel is generally straight, except for the two bends in crossing Choctaw Bar; and this section is being straightened under the present project.

At the lower end of the Bay is an area of 8 square miles with a depth of 24 to 40 feet, forming a safe and commodious anchorage, where it has been the custom of vessels to complete their cargo from lighters, having loaded to their available draft at the docks.

The volume of water discharged into the Bay by the Alabama River system averages about 100,000 cubic feet per second, and the discharge from the Bay during the falling tides is about twice this amount; this outflow kept the outer bar scoured out to a depth of 22 feet without improvement. The current across the bar varies greatly, depending on the direction of the winds, and at times reaches 5 or 6 miles per hour. From the fact that the silt carried by the rivers is largely deposited on the bars at their

mouths, extending them at the rate of 100 feet per year, the outer bar gets little accession from this source.

The natural tendency of this channel from the Bay into the Gulf of Mexico is to move westward, due to storms and normal littoral currents. This is shown in the changes that have taken place since observations began. From 1850 to 1892, the channel moved west 1,500 feet; from 1892 to 1902, this westward movement was 1,000 feet. The width of the 30-foot channel opposite Sand Island light has increased from 1,250 feet in 1850 to 1,900 feet in 1908. Formerly, the western edge of the 30-foot channel moved west faster than the eastern edge, but since 1892 the eastern edge has moved from 20 to 50 feet per year faster than the western edge, showing a rather rapid shoaling and a slow shifting of the channel. The deepest and widest portion of the channel is found off Mobile Point, where depths of 30 to 50 feet extend over a width of 2,000 feet.

The depth of 22 feet naturally existing over the bar has been increased by dredging since 1903, until now there is 28.9 feet at mean low tide, with a tidal range of 1.1 feet.

The following table gives the shoaling since 1903:

Years.	Depth.	Dredged total.	Shoaled.
	Feet.	*Cubic yards.*	*Cubic yards.*
1903–1906	25	288,542	28,000
1907–1908	27	333,565	156,000
1908–1911	29	204,221	61,000

As will be observed, the rate of shoaling was greatest immediately after the storm of 1906 and was probably due to readjustment of conditions following this storm, although ever since 1908 the shoaling has proceeded more than twice as rapidly as before 1907. This 1906 storm submerged Dixie Island, on the east edge of the channel.

Between 1850 and 1892, the outer 30-foot curve marking the Gulf end of the bar in the extension of the natural channel moved out to sea 2,000 feet. Strangely enough, this outward movement has not only ceased, but the 30-foot contour shows a slight tendency to move inshore. The only plausible explanation of the considerable extension of the bar, subsequent to 1850, is that due to some violent storm the entire outer part of the bar was cut off and in the following years gradually was extended to its normal position, which has been maintained from 1892 to the present. As shown by the figures above given, bearing on the condition and location

of the channel through the outer bar, the tendency of the channel westward is clearly established, its width from year to year is uncertain, but at present is growing less; the outer limit of the bar is stationary at present, but judged by the past, is liable to some extension or a considerable shortening as a result of storm conditions.

The channel through the bar follows the usual rule, observed in regard to channels opening into tidal seas, of moving leeward, until the length of channel becomes so great and the rate of flow so checked that a breakout occurs through the windward side in the prolongation of the gorge of the pass from the Bay. Up to the present time no such breakout has been known to occur, as the westward travel of the channel has been slow compared with the similar movement of channels on the Pacific Coast where littoral currents are much more powerful and effective in the lateral movement of sand. There has been only one plan seriously proposed for keeping open this channel through the outer bar, the one followed successfully up to this time, that is: Dredging unaided by jetty construction.

Charts prepared from Weather Bureau records show direction, frequency, movement (total distance of travel) and greatest velocity for a five-minute period of winds at Mobile for the five years ending March 21, 1911. An analysis of the charts shows that the prevailing winds are from the southeast. These winds also show the greatest wind movement, indicating that the storms which produce the maximum sand movement are from the southeast. The total number of storms covered by the tables is sixty-one, lasting from five to one hundred and nineteen hours and having velocities varying from 22 to 47 miles per hour.

Storm directions.	No.	Remarks.
South-East	24	Out of a total of 61 storms shown in the table, all except 6 were from the South-East, North, or North-West, and the remarkable fact is that in all these years there was only one storm each from the South, West, North-East, and South-West.
North-West	21	
North	10	
East	2	
South	1	
West	1	
North-East	1	
South-West	1	
Total	61	

The total wind movement in the five-year period is shown in the table and chart on next page.

Direction from.	Movement
	Miles.
South-East	84,000
North	72,000
North-West	54,000
South	52,500
South-West	30,000
East	18,000
North-East	18,000
West	10,500

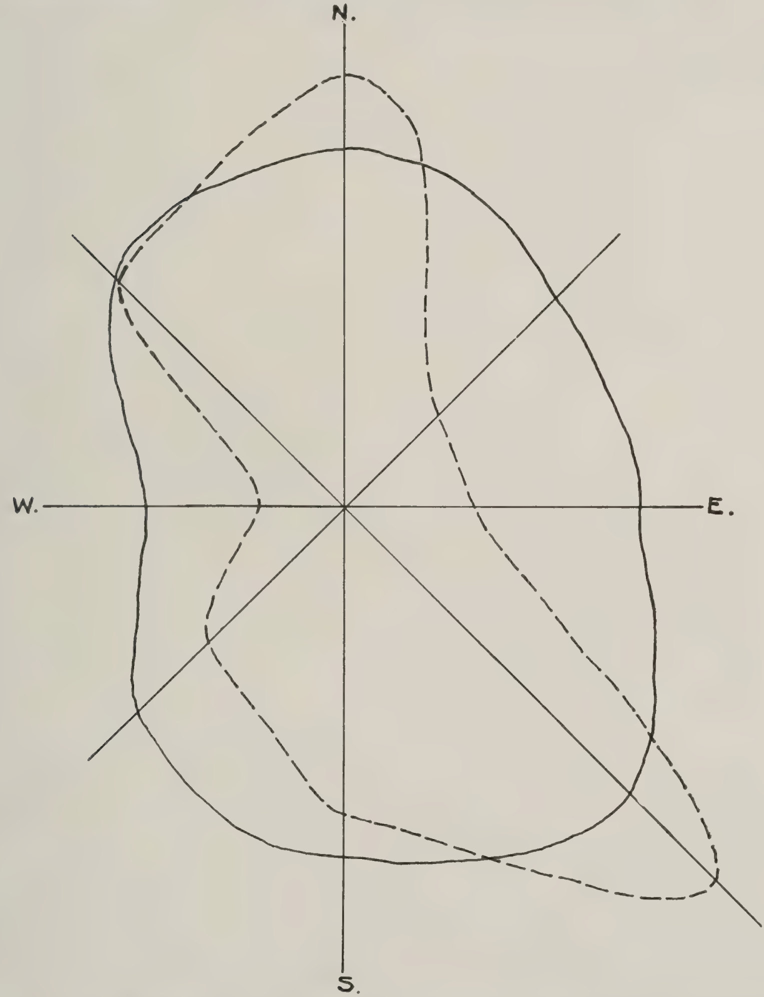

Fig. 2. Curves showing total frequency of winds and total wind movements, at Mobile, Ala., Feb. 3, 1906, to June 27, 1911. Frequency of winds is shown in full line; wind movements are shown in broken line. Scale: Frequency, 1″=600 days; Movement, 1″=30,000 miles.

This summary is conclusive proof of the direction of those winds having sufficient force to cause a considerable sand movement along the Gulf Coast, which lies generally east and west off the entrance to Mobile Bay. These high winds coming from the south-east move the sand westward along the shore and confirm Major Gillette's theory that it is the storm waves that produce the littoral sand movement, rather than the tidal currents which, on this coast, are slight in view of the small mean tidal range, about 1.1 feet. However, when there is a steady on-shore or off-shore wind for several days there is produced a wind tidal range sometimes as much as 5 or 6 feet. It has been claimed by some, notably, Professor Corthell, that there is an eddy current from east to west along this coast, caused by the Gulf Stream flowing eastward from the Bay of Campeachy toward the Florida Peninsula. It is very doubtful whether this current, assuming its existence, would be sufficiently powerful to carry sands; but storm waves breaking in deep water stir up the bottom of the Gulf and throw the sands westward along the shore. The actual conditions found on the Outer Bar as to shoaling and shifting of the channel (noted on page 5) agree with this theory of sand movement.

REQUIRED DEPTH OF OUTER BAR.

The depth of 29 feet now available over the bar is adequate for the needs of all navigation accommodated by the 23-foot channel, but a greater depth will be necessary upon the completion of the 27-foot project. This required depth depends on the wave heights liable to be met under any conditions of weather in which vessels can safely cross the bar. Assuming the greatest height of waves from trough to crest before breaking on bar as 6 feet, the deduction from mean depth for navigation purposes, due to the trough of the waves, would be 2 feet; consequently, for an effective depth of 28 feet over the bar, the channel depth should be 30 feet. But there are several other factors to be considered in determining the required depth of channel, such as maximum height of waves in which vessels can be handled, the "squat" and "pitch" of vessels, and the taking-off of pilots. It is considered that 5-foot waves are the greatest that would be met, on account of the difficulty of handling vessels and removing pilots. The worst storms are from the south-east, with currents parallel to the shore; these striking the current of the channel make the sea rough and choppy, and difficult to navigate. A vessel moving out at full speed draws more water than

when standing alongside of a dock, and this extra draft, called "squat," of the vessel decreases the available depth. Vessels drawing 27 feet would have a squat of about 2 feet. An additional depth is required, due to the pitching of vessels, although for a vessel of 27-foot draft this effect is slight in waves of the height met in crossing the bar.

The depth of channel across the outer bar proposed, in connection with a 27-foot channel inside, is 33 feet, which appears ample for all the needs of vessels making the port of Mobile. The demand for a depth of 35 feet has been made with considerable persistence of recent months, for the purpose of advancing the interests of Dauphin Island as a coaling station. It seems quite possible that in case ample coaling facilities should be installed at Dauphin Island, a considerable number of vessels using the Panama Canal would be coaled in Mobile Bay on account of the cheap coal that can be secured from the Alabama fields and the nearness of this Bay to the Canal. On the completion of the slack-watering of the Alabama rivers, coal could be delivered at Mobile or Dauphin Island at such low rates as to be able successfully to compete with coal barged to New Orleans from Pittsburg.

METHODS OF IMPROVEMENT.

A study of the Harbor with a view to its improvement requires special consideration to be given the currents caused by the streams at the head of the Bay, which, as above stated, tend to the west shore. Theoretically, it would seem that a dredged channel, if in reasonably hard material, would not be difficult to maintain, except at the mouth of Mobile River and in the upper third of the Bay, where the influence of various silt-bearing currents would have an unfavorable effect. Confronted with this difficulty, engineers have prepared many plans in the past for securing the desired depths of channel into the Harbor.

A State Board of Harbor Commissioners, in 1872, proposed to concentrate in Mobile River the flow of all the streams in the Delta and thus scour out and deepen Choctaw Bar (across the mouth of Mobile River) by partially closing Spanish and Tensas rivers, completely closing Pinto Pass,and building a jetty south from the lower end of Pinto Point (fig. 3). The works in Pinto Pass and south of Pinto Point were practically completed in 1872. As had been predicted by a Board of Engineers, considering the whole Harbor situation, these works changed the regimen of Mobile River, set-

ting up a scour that was rapidly filling the channel through Choctaw Bar; and to counteract this tendency Pinto Pass Jetty, as well as 200 feet of the lower end of Pinto Point Jetty, was removed. The results of these jetties emphasized the important principle that to attempt thus to secure a channel through the bar of a silt-bearing river flowing into a practically tideless bay, will result only in forcing the bar farther down into the Bay where the increased area of cross section allows a greater dispersion and consequent slowing up of currents and develops a bar with less depth than when formed farther up. Jetties are usually valuable only where the stream flows into the sea through a tidal basin in which the ebb tidal flow is powerful enough to keep the channel open, or where jetties are required to arrest the sand movement along the shore.

Another experimental step in the improvement of the Harbor consisted in building a series of three spur dikes perpendicular to the shore in Garrows Bend, under the authority of the State Board of Harbor Commissioners. These dikes were built with a view to arresting the inward movement of sand around Choctaw Bar, with the result that during the succeeding years the water shoaled rapidly along the shore in Garrows Bend, but left the channel unbenefited.

Previous to this period of jetty construction there was much doubt felt as to the possibility of maintaining a channel in Mobile Harbor with as great a depth as 22 feet, and in 1880 a project was adopted calling for a channel 17 feet deep and 200 feet wide from the lower Bay to the docks, and an experimental cut 22 feet deep, 100 feet wide, and 300 feet long on Dog River Bar. The result secured in this experimental cut led to the belief that a channel of 22 feet depth with side slopes of 1 on 5 could be maintained with little difficulty.

In addition to the above, the following plans have been suggested in the past for providing for the commerce of Mobile Harbor:

1. Making a harbor on the south side of Dauphin Island (fig. 1) and connecting it with Mobile by rail.

2. Making a harbor on the north side of Dauphin Island, with railroad connections.

3. Making a harbor at Alabama Port with the same connections. (See fig. 1.)

4. Making a canal inland from Mobile to Alabama Port.

5. Damming all but one river, improving that one, and direct-

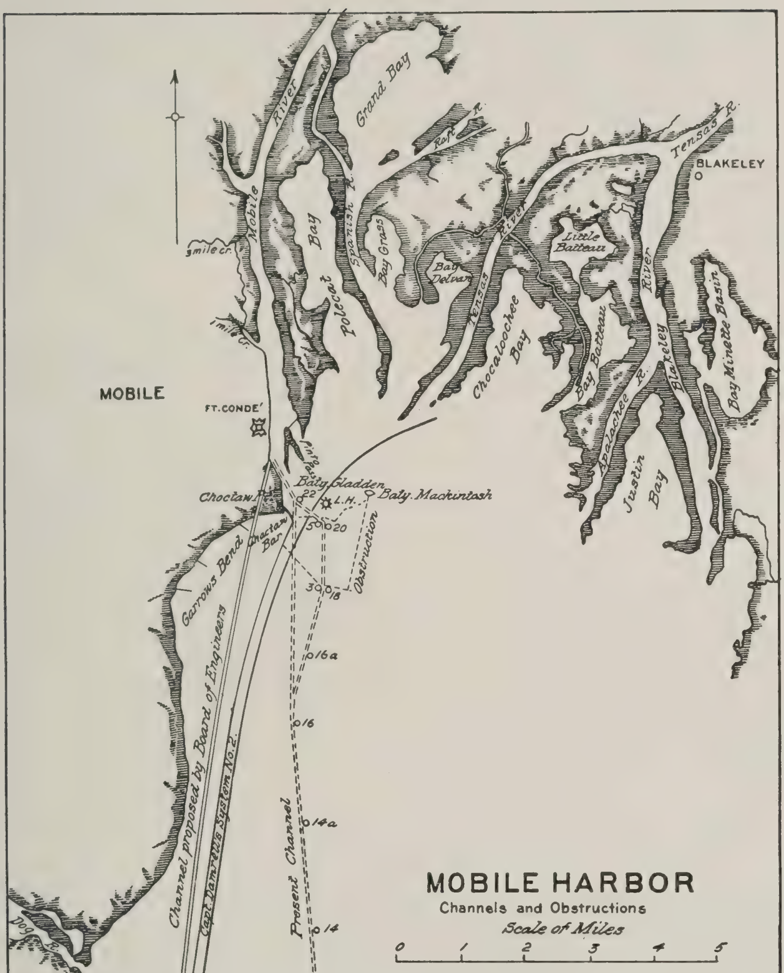
MOBILE HARBOR
Channels and Obstructions
Scale of Miles
0 1 2 3 4 5
MOBILE
FT. CONDE'
BLAKELEY
Mobile River
Grand Bay
Raft R.
Spanish R.
Polecat Bay
Bay Grass
Bay Delvan
Tensas River
Tensas R.
Chocaloochee Bay
Little Batteau
Bay Batteau
Apalachee R.
Blakeley River
Bay Minette Basin
Justin Bay
3 mile cr.
1 mile cr.
Choctaw Pt.
Baty Gladden
L.H.
Baty. Mackintosh
Choctaw Bar
Garrows Bend
Obstruction
Channel proposed by Board of Engineers
Capt. Damrell's System No. 2.
Present Channel
Dog R.
22
50
20
30
18
16a
16
14a
14

Fig. 3.

ings its discharge into the Bay with the expectation of its scouring its way to deep water.

6. Using the tidal current for scouring a channel by training its flow between the entrance and upper end of the Bay between parallel walls.

7. Dredging a channel through the Bay, following the line of deepest water, and carrying the dredged material to a safe distance outside the channel.

8. Dredging and scouring a channel through the Bay by damming all but one outlet, contracted to a proper width, and training the flow between artificial parallel banks to deep water or tidal channel, and dredging out or stirring up all material that would not scour.

It will be observed that plans 2 and 3 are still contemplated, but by private corporations, viz: The Dauphin Island Company and The South Mobile Improvement Company. The former proposes a 30-foot channel from the main ship channel into a dredged basin between Little Dauphin and Big Dauphin islands, where docks are to be built with special facilities for a coaling station. The South Mobile Improvement Company has commenced dredging a channel and building wharves at Alabama Port. Both these projects involve railroad connection with Mobile. Plan 5 (jetties and dams) was tried unsuccessfully by the State Board, as above described. No. 6 was out of the question, on account of its great cost and the difficulties to be met on account of the changed regimen of the river thus extended to the lower Bay. It has been found that the channel would require a slope of 1.8 feet per mile or a rise of 43.2 feet from its mouth to Mobile, a condition evidently impossible of attainment, since Mobile is only about 20 feet above Gulf level. No. 7 is the plan that has been followed continually from the beginning of improvement, while No. 8 is a combination of Nos. 6 and 7. Each of these plans has had at various times its ardent advocates, some of whom have been prominent engineers. The main objection to the adopted dredging plan was that it would be impossible to maintain economically an open dredged channel in the middle of a large shoal bay, receiving vast quantities of sediment, and subject to a complicated system of river and tidal currents.

The fact that the dredged channels described below have been economically maintained at all depths secured, demonstrates the

wisdom of the plan followed up to this time. No shoaling of any consequence took place in the Bay or on Dog River Bar from 1828 to 1878, so far as the records show. A 10-foot channel was dredged in 1838-1839 across Choctaw Bar; this refilled at the rate of 1½ inches per year in the twenty-one years up to 1860. It has been found that the side slopes of the dredged channel, as measured one and one-half years after dredging, assume a slope of 1 on 5. In the 27-foot channel there will be about 1,000,000 cubic yards of shoaling per year, requiring about $100,000 per year for maintenance, not an excessive amount for a channel nearly 30 miles long and some 17 feet deeper than the surrounding bottom of the Bay, especially considering the light character of the surface material (1.3 specific gravity).

WORKS OF IMPROVEMENT.

Between 1826, when the first appropriation by the General Government was made, and 1857 a channel 10 feet deep was dredged through the shoals in Mobile Bay up to the docks. Between 1870 and 1876 this depth was increased to 13 feet, the channel being dredged to a width of 300 feet through Choctaw Pass and 200 feet through Dog River Bar. Between 1880 and 1886 a channel 17 feet deep and 200 feet wide was secured; and from 1888 to 1896 a channel 23 feet deep with a top width of 280 feet was completed from the Gulf of Mexico to the mouth of Chickasaw Creek, 3 miles above Mobile. The total amount expended up to 1899 was $3,648,630.06, of which $115,000 was for maintenance.

In March, 1899, a project was adopted for a 23-foot channel with a bottom width of 100 feet from the entrance of the Bay to Chickasaw Creek. This project was completed in 1909 at a cost of $1,896,860.58, of which $610,832.07 was applied to maintenance. The outer bar was made a separate project in 1905 and a channel authorized 30 feet deep and 300 feet wide; at the present time there is a channel about 29 feet through the outer bar, at a cost of $173,004.55, of which $77,000 was for maintenance. The present project for Mobile Harbor, adopted in 1910, provides for a channel 300 feet wide in Mobile River, 200 feet wide in the Bay, and a turning basin at Chickasaw Creek, 600 by 800 feet, all with a depth of 27 feet.

Recent progress in dredging methods is illustrated by the decrease in prices bid for the work in Mobile Bay. In 1872, a contract was let for the dredging through Choctaw Bar at 30 cents per cubic

yard, bids ranging from 30 to 40 cents. In 1880, bids were received ranging from 12.3 cents up to 19.4 cents per cubic yard. In the last bids, opened June 27, 1911, the award was made for one section in the Bay as low as 5.38 cents per cubic yard and 9 cents per cubic yard the river. This decrease in cost has been brought about by improvements in dredging machinery, but more especially by the competition of Government owned dredges.

THE OBSTRUCTIONS.

Early in 1861 the Confederate forces garrisoning the defences of Mobile constructed two rows of pile obstructions near the mouth of the rivers (fig. 3). Openings were left in the obstructions for the passage of vessels up Spanish River channel and Mobile River channel; these openings are shown on the map.

The openings were protected by five 10-inch Columbiads and two 7-inch Borrk guns in Battery Gladden at the channel elbow and in Battery McIntosh at the mouth of Spanish River, both built on artificial islands. No effort was made by the Federals to pass the obstructions, but a fleet of gunboats came up to them after the fall of Forts Morgan and Gaines and threw some shells into the city.

The effect of these obstructions on the channel is not definitely known, though they probably assisted in turning the flow of the river into the dredged channel and caused a more rapid accumulation of driftwood and sand on Choctaw Bar. The snagboat *Demopolis* has recently removed a large part of both rows of obstructions on the west side of the channel, in preparation for the proposed cut-off.

LOGS AND DEADHEADS.

No sack rafting or loose log driving is allowed in Mobile River under Section 15, Act of March 3, 1899, which provides " . . . that it shall not be lawful . . . to float loose timber and logs in streams or channels actually navigated by steamboats in such manner as to obstruct, impede, or endanger navigation." Nevertheless, there has been much complaint in the past, caused by "sinkers" from poorly constructed rafts. The principal variety of timber now forming the bulk of the export trade from this port is loblolly pine, which, when soaked, will barely float submerged, and consequently forms the most dangerous obstruction. It is often found that these timbers, when recovered by the snagboats, are full of

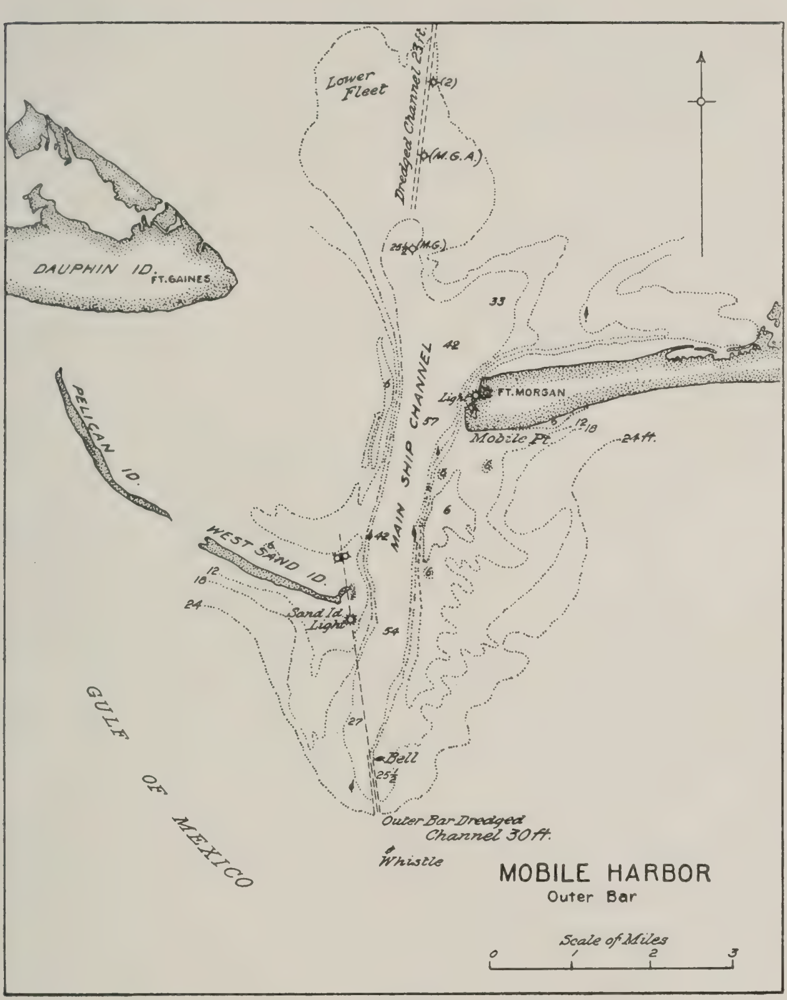
Lower Fleet
Dredged Channel 23 ft.
(2)
(M.G.A.)
(M.G.)
DAUPHIN ID.
FT. GAINES
33
42
MAIN SHIP CHANNEL
Light
FT. MORGAN
57
Mobile Pt.
24 ft.
PELICAN ID.
WEST SAND ID.
42
54
Sand Id. Light
27
Bell
Outer Bar Dredged Channel 30 ft.
Whistle
GULF OF MEXICO
MOBILE HARBOR
Outer Bar
Scale of Miles
0
1
2
3

Fig. 4.

cuts made by propeller blades and often pieces of the blades themselves are imbedded in the logs, showing the constant danger to vessels.

SEWAGE DISPOSAL.

Mobile has a system of gravity sewage completed in 1900 at a cost of $400,000, consisting of 50 miles of sewers 8 inches to 30 inches in diameter. There are ten outlets into Mobile River at the foot of the following streets: Lipscomb, St. Michael, Dauphin, Conti, Government, Church, Monroe, Eslava, Charleston, and Delaware. These sewers carry all waste and surface water of the city into the river, but up to the present time the amount of waste has not been in sufficient quantity to noticeably defile the river.

COMMERCIAL.

Mobile has the following railroad systems, giving ample connections to all parts of the country:

Mobile and Ohio Railroad Co., 926 miles; Southern Railway Co., 7,496 miles; Louisville and Nashville Railroad Co., 4,349 miles; New Orleans, Mobile, and Chicago Railroad Co., 402 miles.

In addition to the railroad connections, Mobile has packet lines running to Montgomery and other points on the Alabama River, and to points on the Warrior and Tombigbee rivers. It is generally thought that barge lines will be established between Mobile and the Alabama coal fields on the completion of the canalization of the Warrior and Tombigbee rivers. This, however, is open to doubt, as up to this time no definite steps have been taken in the direction of handling coal by barges, although the project for improvement is well under way. It seems, however, that the barging of coal must inevitably become an important traffic, in view of the low cost of delivering coal by barge on board ships at Mobile, conservatively estimated at $1.25 per ton, including profit on mining. In the Warrior coal fields there are more than 50,000,000,000 tons of bituminous coal eventually available to this river transportation.

Mobile has ample foreign connection by means of regular lines of steamers to European, Asiatic, and South and Central American ports, besides large numbers of tramps going to ports all over the world. In the calendar year 1910, 540 steamers with a tonnage of 639,796, and 179 sail vessels with tonnage of 79,070 cleared from the port of Mobile. The number of vessels entering the port has shown a considerable decrease in the past fifteen years,

due partly to an increase in the tonnage of vessels used during that period; but since 1905 there has also been a decrease in tonnage.

TERMINAL FACILITIES.

Mobile has a river frontage of 24,800 feet on the west side of the river; of this, about 16,000 feet are improved or easily capable of improvement. Five railroads own 4,087 running feet of improved frontage; the city owns about 1,000 feet and private citizens own the remainder.

Of the improved property owned by the railroads, the Mobile and Ohio owns 1,160 feet, equipped with piers, slips, warehouses, and grain elevator, with docking capacity of 2,400 feet; the Southern Railway has 320 linear feet, equipped with coal hoists, piers, slips, and warehouses, with docking capacity of 2,400 feet; the Louisville and Nashville Railroad owns 1,500 running feet, equipped with shed, with a docking capacity of 1,500 feet; the New Orleans, Mobile, and Chicago Railroad Company owns about 1,000 running feet, equipped with piers, with a docking capacity of 2,000 feet. The property of the city is equipped with sheds and has a docking capacity of 1,500 feet. There is some water front property improved by private individuals, but not equipped with warehouses, and having no facilities for general trade. All the property on the east side of the river is privately owned, and is now undergoing extensive development.

The Mobile and Ohio Railroad has on one of its piers a grain elevator with storage capacity of 250,000 bushels. The railroads entering Mobile have provided connections with many of their own piers, and freight is delivered direct from the railroads to such piers. The Mobile and Ohio, the Southern Railway, reach four piers to which their rails deliver freight, besides a coal pier. The Louisville and Nashville and the Mobile and Ohio have tracks into the fruit shed. The Louisville and Nashville also have tracks into the pier of the Mobile Coal Company, and this road has track along its entire 1,500 feet of water frontage.

The Hieronymous dock has a docking capacity of 1,400 feet and the Turner-Hartwell docks have a docking capacity of 3,800 feet; these are open to all business on equal terms, except that the cargo to Turner-Hartwell docks over certain railroads is charged fees by the railroads. The railroad wharves are open to general traffic when it does not interfere with the steamship lines. Re-

cently the matter of switching charges has been investigated by the Interstate Commerce Commission and orders issued to place all docks on an equality and open to the public on equal terms.

A portion of the 1,500 feet of property owned by the city is rented to the Mobile and Ohio Railroad Company; a portion is used by river boats, the remainder is unimproved.

It is stated that the charges at Mobile are in excess of those at Gulfport for harbor fees, pilotage, and tonnage. As an illustration, to unmoor a vessel from up the river, place it in a slip and return to mooring costs $140.00, of which the deputy harbor master gets $20.00 for being present at the ceremonies of unmooring and mooring; the tug gets $70.00 for standing by during these operations; the remainder is for docking and towing. In a tabulated list of expenses for pilotage, quarantine fees, wharf charges, harbor master's fees, State tax, etc., of the principal cities of the United States, given in a pamphlet entitled, "Mobile: Her Trade, Commerce, and Industries, etc." published some years ago, these charges were shown to be less at Mobile than any other port. A large fruit importer, Mr. Oteri, of New Orleans, La., was quoted in the *Times-Democrat*, in 1898, as giving the following cost at New Orleans and Mobile for a 500-ton steamer:

	New Orleans.	Mobile.
Wharfage	$70.00	
Harbor Master	10.00	
River Pilot	30.00	
Bar Pilot	70.00	$35.00
Quarantine Physician	35.00	7.00
Unloading 10,000 bunches bananas	300.00	200.00
100 tons of coal	330.00	250.00
30,000 feet of lumber	300.00	130.00
Miscellaneous freights	55.00	10.00
Unloading 10,000 cocoanuts	100.00	65.00
Telegrams from Jetty	2.00	1.25
Custom Broker	7.50	4.00
Total	$1,309.50	$702.25

Present comparative charges are not so favorable to Mobile, except on the item of wharfage, as shown in the table on the next page.

Comparative Port Charges at the Gulf Ports.

Port.	Pilotage.	Wharfage.	Harbor Master.	Shifting—	Towage.
	Feet. — *Per Foot.*		*Per Shift.*	*Per Shift.*	
Mobile	9 — \$2.50 $9\frac{1}{2}$–10 — 3.00 11–12 — 3.25 $12\frac{1}{2}$–14 — 3.50 $14\frac{1}{2}$–20 — 5.00 $20\frac{1}{2}$– — 6.00 Extra, when towed, fifty cents per foot.	None (except on cargo)	\$10.00 average of 2 to 3 shifts.	\$25.00	10 cents per registered ton.
	On Deepest Draft.	*Per day per gross registered ton.*			
New Orleans.	Bar, \$4.00 per foot River, \$1.50 per foot	First 3 days, 2 cents. 4 to 6 days, 1 cent. 7 to 36 days, 0. Sheddage, $\frac{1}{2}$ cent.	In ballast \$10.00, with cargo \$15.00		
	On Deepest Draft.				
Galveston	\$4.00 per foot	None (except on cargo)	None	\$35.00	
	Feet. — *Per Foot.*				
Pensacola	6–10 — \$3.00 10–14 — 4.00 14–20 — 5.00 20– — 6.00	Per day per net registered ton, 1 cent. (Sundays and holidays excepted.)	None		$1\frac{1}{2}$ cents per net registered ton.
	Per Foot.		*Per Shift or for Moving.*		
Gulfport	All drafts, \$3.50	None	\$5.00	\$10.00	10 cents per ton, less 10 per cent.

At Gulfport, the charges are less on all items than at any of the other ports, except on towage. The low charges at Gulfport have been due, probably, to a desire to build up this port for the benefit of the Gulf and Ship Island Railway. Mobile has been at a disadvantage also in the matter of terminal charges on shipment through Mobile from the inland cities. For example, the Southern Railway and the Mobile and Ohio Railroad refuse to absorb the terminal and switching charges on these car-load shipments, unless they are to vessels docked in the slips of the Mobile Docks Company and to the Mobile Liners, Inc. As an example, the following table shows how the railroads in reality give rebates on shipments to their favored connecting docks and favored cities:

	New Orleans-Participated in by M. & O. and Southern Ry.	Galveston.	Mobile. (L. & N. and N. O., M. & C. when to competing docks)	Mobile. (M. & O. and Southern when to competing docks)
Grain, carload, minimum weight 40,000 lbs., at 16½ cents per 100 lbs.	$66.00	$66.00	$66.00	$66.00
Less terminal charges absorbed by railroads—				
Switching, per car	$2 to $7	$1.75	$2.00	--------
Transferring or unloading	$4.00	$2.50	$2.50	--------
Wharfage, paid by vessel	-----------	$4.00	$2.00	--------
	$6 to $11	$8.25	$6.50	--------
Net earnings to railroads	$55 to $60	$57.75	$59.50	$66.00

This practically forces the independent docks out of business and drives competing lines of vessels from the port, because all switching and terminal charges are absorbed on shipments to Galveston, Gulfport, and New Orleans. This discrimination is due to an arrangement of the Southern and Mobile and Ohio railroads with the Mobile Liners, Inc. and the Mobile Docks Co., as against the independent steamships and docks, including the city docks, and has been the subject of a recent investigation by the Interstate Commerce Commission.

Comparative Table of Wharfage Charges at Gulf Ports. (In cents.)

	Mobile.	Galveston.	New Orleans.	Pensacola.
Bagging, per 100 pounds	2	1½	1½	2½
Bananas, bunch	1	½	0	0
Brick, per 1,000	25	30	15	--
Cement, per 100 pounds	1	1¼	½	1
Coffee, per 100 pounds	1	2½	--	--
Cotton	3	1½	3	1½
Cotton-seed hulls	1	1¼	4½	--
Cotton-seed	1	1	4½	--
Fertilizers, per 1,000 pounds	25	25	15	15
Grain and grain products	1-3	½	½	--
Sisal	1¼	3½	3½	--

PROJECT FOR IMPROVEMENT OF MOBILE HARBOR.

The present project provides as follows:

"*a.* The formation of a dredged channel 27 feet deep at mean low tide water, 200 feet bottom width in the Bay and 300 feet bottom width in Mobile River with side slopes 1 on 5.

"*b.* The formation of a straight cut-off from Beacon 22 to Beacon 16 (see fig. 1, p. 3).

"*c.* The formation of a suitable turning basin, 600 by 800 feet, at the mouth of Chickasaw Creek.

"*d.* The construction and operation of two hydraulic dredges, one of which, the *Wahalak,* commenced work in March, 1911.

"*e.* The completion of the work in four years from July 1st, 1910, at an estimated cost of $1,802,548, including $100,000 per year for maintenance."

This project follows the old channel, except for the cut-off between Beacons 16 and 22, and was selected from three alternative locations surveyed in 1909. According to the data available, the selected channel will probably be the one most easily maintained; it is also straightest and shortest of the three, involving a direct cut-off between Beacons 16 and 22 (see fig. 1, p. 3). Channel No. 2 contemplated a cut-off from Beacon 18 to 22, with greater curvature than the one selected (No. 3) but straighter that the old channel (No. 1).

The straight channel will facilitate navigation through the Bay and to an extent relieve ship owners from the inefficiency of pilots, through whose neglect or ignorance vessels are frequently run aground at the bends of the channel. Channel No. 3 is 1,241 feet shorter than the present one, and accordingly that much less to maintain. It is questionable whether the cut-off is more or less

expensive to maintain than the corresponding portion of the present channel, for there are arguments on both sides. More water from Mobile River will flow in the cut-off with less tendency to shoal, but there will be less flow received from Spanish and Tensas rivers. Moreover, the old channel was naturally deepest, indicating ease of maintenance here; but the obstructions placed across Choctaw Bar in the Civil War probably did much to keep the old channel open through the gaps left for that purpose. Between 1903 and 1909, it cost an average of $5,000 per year to maintain the section from Beacon 16 to Beacon 22 (one-tenth of the total length of channel), whereas the maintenance cost for the entire channel was about $90,000 per year, indicating a considerably smaller proportionate cost on this section than on the rest of the channel. A dike on the east side of the channel would probably decrease the maintenance cost, but the advisability of such a work can not be predicted until experience shall have been had with the new channel.

The bottom width of the channel should be increased to 300 feet as a part of the maintenance work after completion of the present project. In a channel of 200 feet width it is difficult for two fully loaded vessels to pass without sheering, due to suction or to the nearness of the vessel to the bank of the channel. This is especially true of the vessels of 50 to 60 foot beam which will use the improved channel. A light draft vessel can hug the bank and thus pass a heavily loaded one with safety, and since the incoming vessels at the present time are generally not loaded or partly loaded, the present channel width causes little trouble. As imports increase, the necessity for a wider channel will become more pressing. This increase of width can be economically secured in connection with the maintenance work, because one large capacity dredge will be constantly required for this work alone, and a portion of its time can be devoted to enlargement.

The width of 300 feet is adequate for the present commerce, but with the increasing length of vessels using the Harbor, say, up to 400 feet, it is evident that 300 feet width gives a small margin for turning, warping into slips, and for passing several small craft at once. The harbor lines are from 700 to 800 feet apart, so that a 500-foot channel would leave only 100 to 150 feet to be dredged by private interests in front of the slips on each side of the river to give a commodious harbor throughout.

The material to be dredged in maintenance comes into the chan-

nel in one of the following ways: 1. Shifting silt of the general bay floor, due to tide and wind currents; this is independent of the width of channel. 2. Material ploughed in or sucked in by vessels running near the side of the channel; this is greater for the narrow channel on account of the necessity of vessels running closer to the banks in passing. 3. Deposits from the rivers; this will be greater for the wider channel, due to the greater decrease of velocity of the increased amount of water bearing silt. 4. The return to the channel of material deposited by dredges; this is practically independent of the width of the channel and depends on the distance to which material is carried for deposit. It is thought that where this material is deposited on the west side there will be little tendency to reenter the channel. Taking all causes of shoaling, the total yards of material to be moved in the total maintenance work is about the same for both channels. This view is supported by past experience in the maintenance work. As an instance, in the lower Bay the old channel was dredged to 150 feet width at bottom to obtain suitable side slopes in the very soft material found there; in the upper Bay the width was 100 feet, and yet in the lower portion the shoaling was less than 85 per cent of that in the upper portion.

METHOD OF MAINTENANCE.

The two essential considerations in designing a maintenance plant are (1) low total cost and (2) small hindrance to navigation by the working plant. Under the head of cost, must be considered the original cost of dredge, tender, and attendant equipment; operation—including depreciation, interest charges, ordinary repairs and overhauling.

There are three types of dredges possible for this work: the spud dredge, such as the *Wahalak*, the sea-going hydraulic dredge, and the Fruhling type. Spud dredges are most common in the development of the harbors of the United States, both for original improvement and for maintenance. They are very efficient for improvement work where the distance the material must be pumped is about 1,500 feet or less. For maintenance work the conditions are not so favorable for the use of these dredges. Being permanently attached to a pipe line on one side, it is necessary for passing vessels to take the other side and in narrow channels to move very slowly to avoid injury to the spuds or pipe line, due to suction. The dredge must ease off on its breast and quarter

lines with the loss of a certain amount of time. Where the dredge is working on the side of the channel opposite the pipe line a vessel of deep draft can only pass when the dredge stops work and shifts over on the pipe line side or else opens the pipe line for the vessel to pass. This objection does not apply to a great extent in Mobile Bay, because any widening of the channel that may be done must be on the side toward the dump on account of the position of the beacons near the opposite side.

Maintenance work in general requires the removal of a thin layer of material over the bottom of the channel and thus necessitates a constant shifting of the pipe line. This is a serious objection in the upper Bay on account of the difficulty of this shifting in the shallow water found outside the channel. Another disadvantage of the spud dredge is that it is more unwieldy than the self-contained type and in storms of any severity is liable to suffer injury to its pipe line. This has occurred on more than one occasion in Mobile Bay. One of the advantages of the spud dredge is that the work can be done very thoroughly, leaving no ridges or lumps, since the distance of each swing forward can be regulated with accuracy to the capacity of the pump. The spud dredge can not be used advantageously on the bar channel because of the impracticability of using the pipe line in the rough seas usually found there, and a considerable amount of time would be lost in waiting for suitable weather to take the dredge out to the work. On the other hand, a hopper dredge is not suitable for the upper third of the Bay channel on account of the shoal water on each side necessitating a haul of 5 to 7 miles to dump.

Hydraulic hopper dredges may be self-contained with their own propelling machinery or depending on a towboat to haul them along while material is pumped into the hoppers, and to tow them to and from the dump. The self-propelling type is cheaper in first cost, due to the fact that no attendant plant is required. A smaller working force is needed for the sáme reason, the men are all on one vessel and can be used on different classes of work, whereas the crew of the tow boat are not available for work on the dredge. The self-propelling type is easier to steer straight along the channel than the towed type, which is with difficulty kept in a straight line or put in a desired position. Both of these have the disadvantage of requiring water between the channel and dump from 10 feet upwards in depth. A certain amount of time is lost by a hopper dredge going to the dumping ground, varying with the distance,

whereas the spud dredge works constantly, except for delays due to shifting or break in the pipe line. The hopper dredge is most valuable in handling sand or gravel and least economical in moving light silt. The pipe line dredge works readily in all classes of material. In the upper portion of the Bay the material is largely sand, but the greater part of the channel is blue clay and silt of very light specific gravity and difficult to handle in hoppers. The sea-going type is not liable to injury in storms and can be quickly brought to a place of safety when a storm approaches. This type is the best available for maintenance or improvement work on the bar.

The Fruhling dredge is the most recent development in subaqueous excavation machinery and has some advantages over all other types of dredges, especially in removal of soft mud so frequently found in maintenance work in our harbors. In general, the Fruhling dredge is a sea-going hopper type, equipped with dredging head of special design. The hull is divided for a part of its length aft by a well in which a heavy girder attached to a cross trunnion at the upper end carries a suction pipe, a pressure pipe and a suction head or plough from 10 to 20 feet in width perpendicular to the axis of the suction pipe. This head is the distinctive and patented feature of this dredge, the remaining parts are similar to those of other hopper dredges.

The action of the dredge is as follows: The outer end of the girder with its pipes and suction head is lowered to the bottom and the dredge steams ahead at a fair speed of, say, 4 or 5 knots (depending on the material being dredged), dragging the heavy head with its sharp cutting edge through the surface of the bottom. A section of the material is cut by the head and forced into the same, completely filling it and holding back the surrounding water. As this material is pushed forward into the head it is stirred up, if necessary, by jets of water under high pressure from a pump in the dredge. The material is thus diluted sufficiently to be carried by the pump into the hopper. The effect of the box-like head is to exclude all water from the pumped material, except sufficient to give the fluidity required for its efficient pumping. The ordinary cutter-head dredge pumps up from 10 to 20 per cent solid material; this dredge raises 60 to 90 per cent solid material in mud and 20 to 50 per cent in sand. The large increase in efficiency is due to the lost energy of lifting water.

Up to this time only one of these Fruhling dredges has been

built in the United States, for use at the mouth of the Mississippi River. It is the largest dredge of this type ever built, having a hopper capacity of 3,000 cubic yards, and a guaranteed dredging capacity of 4,000 cubic yards of mud or 2,000 cubic yards of sand per hour. The first cost of a Fruhling, with complete equipment of a capacity of 800 cubic yards per hour actual dredging time or about 500 yards per hour effective working time, is around $200,000. This is about the same as for a spud dredge of like capacity. A sea-going dredge of the usual type, of equivalent capacity, would cost about $350,000.

Figures on the cost of operation of Fruhling dredges are not obtainable, except from literature of the company owning the dredge patents; from these it appears that maintenance work like this in Mobile Bay can be done at a gross cost slightly less than that of the spud dredges, which runs about 3.5 cents gross per yard. Taking all features of cost and convenience of operation into account, I conclude that a spud dredge is best suited for the upper 9 miles of Bay channel in maintenance and widening; that either the spud or Fruhling dredge is suitable for the lower Bay channel, with a slight advantage in favor of the Fruhling; that the Fruhling dredge is best suited to the work of maintenance and widening on the bar. I therefore propose for maintenance work a Fruhling self-propelling hopper dredge, equipped with pipe line for use in the upper part of the Bay, where the water is shallow outside the channel, and in the river: while the hopper will be used on the bar and in the lower Bay, where the depth outside the channel is sufficient for dumping within a distance of three-quarters of a mile. Taking the whole Mobile District, there are now two high class spud dredges available for maintenance work on the channels at Mobile, Pascagoula, and Gulfport. A Fruhling dredge at a cost of $200,000, added to this equipment for use especially on the Mobile, Horn Island, and Ship Island bars, and to assist in channel maintenance, would give a very complete equipment for the District.

COST OF MAINTENANCE AND ENLARGEMENT.

It has been estimated from the past history of the harbor that 1,000,000 yards of material will come into the channel yearly. To remove this at the rate of 200,000 yards per month will require five months per year, leaving seven months for maintenance of the bar channel, widening the channel, and laying up for overhauling and repairs. A safe estimate for this time of laying up is one and a half months annually, leaving five and a half months available.

The annual shoaling on the bar averages less than 25,000 yards, which could be taken out in two weeks time at the outside. It would, however, be well to put in one month's work here annually until the channel of 33 feet depth and 300 feet width is secured. The present excavation required to secure this channel is about 200,000 yards, which should be taken from the eastern edge of the present channel in order to hold this edge so that, as the natural scour erodes the west bank, the channel will be gradually widened, while maintaining its present location. This location of the bar channel conforms very closely to the theoretically correct location perpendicular to the direction of prevailing winds, and hence is easier maintained than a channel at any other position. The difficulty of working on the bar, measured by the cost of dredging in the past, is about twice as great as in the Bay channel.

After taking out the time for maintenance of bar and other channels and for overhauling, there would be left four months available for widening the channel in Bay and river. The present project requires the removal of 16,623,600 yards; the project herein proposed (500 feet width in river and 300 feet in the Bay) requires 28,808,200 yards; so that there would be 12,144,600 yards to be excavated in enlargement. At the rate of 200,000 yards per month this would require about sixty months, or fifteen years, to complete with one dredge, while maintaining the present project channel. If the *Wahalak* should still be available on completion of the present project, the two dredges together could complete the enlarged channel in addition to maintenance in six years at a cost of $600,000, or about 5 cents per yard. In view of the authority contained in the Act providing for the Mobile project to purchase two dredges, and considering the low cost of the work already done, a Fruhling dredge might be purchased for use in completing, maintaining, and enlarging the channel.

SUMMARY OF PROJECT—BY AUTHOR.

a. Complete present project.

b. Purchase of a Fruhling dredge of 800 yards capacity per actual dredging hour at a cost of $200,000 (out of funds for present project).

c. Widen the river channel to 500 feet, the bay channel to 300 feet, 30 feet depth, and the bar channel to 300 feet and 33 feet depth, at a cost of $600,000.

d. Maintain the channel projected at a yearly cost of $100,000.

e. Operate the Fruhling dredge and the *Wahalak* in maintenance and enlargement of the channels.

Notes on Search-light Mirrors at the Engineer School

BY

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One of the problems undertaken at the Engineer School some time since was the attempt to develop a search-light mirror having the qualifications necessary to allow of successful and continued use of search-lights under the rough conditions of field service. The desiderata of such a mirror are:

1. Good illuminating power—
   a. Capable of high polish of reflecting surface;
   b. Absence of light-absorptive qualities in the material.
2. Permanence of reflective surface.
3. Ruggedness against shock in handling and transportation.
4. Mechanical perfection of form, and permanence of this form under great temperature variations occurring during operation.
5. Continuance in service when injured by missiles.
6. Ease of cleaning and absence of liability to damage by unskilled cleaners.
7. Lightness of weight.

The above are desirable in any mirror whether for permanent fixed lights, hand portables or field lights; but they are especially necessary for field service, particularly that quality of ruggedness under the shocks and strains of transportation over rough roads and rougher fields.

Several classes of mirrors of 24-inch and 30-inch diameter have been tested under operating conditions. These mirrors comprise a 24-inch parabolic glass mirror, a 30-inch monel metal mirror, a 30-inch gold surface mirror, and a 30-inch segmental glass mirror.

GOOD ILLUMINATING POWER.

The glass mirrors are found to have, as might be expected, the

highest illuminating power, since the glass on which the silvering is deposited is capable of an extremely perfect surfacing, and the silvering applied to it is much more brilliant in its contact surface than it is possible to obtain by mechanical polish of the metal. So marked is the difference that it was found that the 24-inch glass mirror, though old and defective in spots, was the equal of the 30-inch monel metal mirror and the gold mirror in illuminating power. The gold metal mirror, as compared with the monel metal, reflected a light which brought out the objects illuminated by it in more nearly their natural and daylight colors, and hence is more pleasing and restful to the eye and aids the observer in his work. On the other hand, flaming of the arc or other defects in the arc which causes wavering spots of dark or colored lights to sometimes occur in white beams and which are not particularly objectionable if only sporadic, are greatly accentuated in the yellow beam. It is claimed that the yellow beam has greater penetration in fog, but this has not been confirmed at the Engineer School. The fact that the yellow beam is inferior to the white beam is explained (other features of the mirror being the same) by the fact that the component colors making up the total beam are materially reduced by reflection from a yellow surface which absorbs all but the yellow rays. This can not be strictly true, however, since the mirror is not as pronounced a yellow when brilliantly illuminated as when in ordinary light, and consequently other rays must be reflected from the highly polished *surface* and not be absorbed by the *material*. It is difficult to distinguish the proportions of light lost by absorption of the material and that lost by poor surfacing. But the material absorption is far less in importance than the absorption due to poor polish, as is evident from the increased power of the more highly surfaced reflecting surface of the glass mirror. The segmental mirror has bands of metal crossing it, which reduces the effective reflecting area. However, the ineffective area is relatively small and the mirror, when compared with the 24-inch glass mirror, was found to have about the increase in lighting power that one would expect, and much greater power than either of the metal mirrors of the same size.

PERMANENCE OF REFLECTING SURFACE.

In the glass mirrors deterioration occurs through minute cracks occurring in the silvering and, finally, separation of the silvering from the glass, but the remedy is simply a resilvering. It is not

necessary to risk change of form through repolishing or regrinding; and any hand cleaning to be done is due to dirt on the front of the mirror which can be readily removed, and, since the surface of the glass is hard as compared with the metals, with comparatively little risk of damage if reasonable care is used. Corrosion of the reflecting surface or of the glass does not enter into the problem during the expected life of the mirror. The monel metal, though the makers claim absolute non-corrodibility, was found to be, after about two years, covered with a network of very fine cracks, which appeared to be due to separation of crystals, as the mirror surface had a crystalline appearance; in addition, frequent cleaning of the surface was found necessary as the metal tarnished. The gold mirror, too, was found to require removal of the tarnishing at frequent intervals, although there was apparently no corrosion whatever.

RUGGEDNESS.

It is evident that glass, because of its nature, is much more liable to damage by shock than is metal, and for this reason the metal mirror presents great advantages for field service over the glass mirror. But if the glass mirror be cut up into pieces which are, in turn, supported by a metal framework, there results a certain amount of flexibility which renders the mirror less liable to damage. The fact, too, that, in general, the entire mirror will not be smashed by a blow, but only one or two segments, leaves a considerable area of the mirror intact for temporary operation, with the possibility of replacement of the damaged segments with new ones.

MECHANICAL PERFECTION OF FORM AND PERMANENCE OF THIS FORM.

Metal is much less stiff than glass, and since mirrors of metal must be very thin in order to avoid too great weight, the danger of giving under the process of grinding and polishing is much greater and the consequent distortion is greater. Glass, too, is much harder, and hence can be shaped more perfectly than metal; the principal trouble arising in the use of glass being striation, but this is not appreciable with the modern perfection of lens-glass manufacture. The cutting of a glass mirror into segments after being ground is apt to allow distortion due to the unbalancing of internal material stresses, but this the makers deny. Then, too, it is very difficult to produce two perfectly identical mirrors, and to do this is necessary if there are to be spare segments. The placing of the segments in the framework is a matter of the nicest skill, if

the mirror is to resume a perfect shape. This seems to offer the greatest possibility of failure of the segmental mirror. The light from this mirror was turned, at a range of about 100 feet, on a light-colored building, and the shape and uniformity of the lighted area noted. The area was not perfectly circular in form, but slightly distorted in a 2 o'clock direction. In addition, there was noticed the faint trace of rectangle, the diagonals of which were horizontal and vertical, and the corners of which projected very slightly beyond the normal circle. These peculiarities were not, however, noticeable except when pointed out, and the functioning of the mirror did not seem to be affected at all when used at longer ranges. At 400 yards the circle of illumination was equal in definition and apparent uniformity to the circles from other mirrors. No replacing of segments has been done. (See translation at end of article in regard to replacing segments.) No troubles due to great temperature variations have been experienced with any mirrors at the School, but it is known that mirrors have been cracked by the intense heat of the electric arc. This trouble does not occur with metal mirrors, and it is claimed that the segmental mirror also overcomes this danger.

INJURY BY MISSILES.

All the advantage lies with the metal mirror in this regard, since a hole would simply be cut out of the mirror, and if the remaining velocity were high, only a very small area surrounding the hole would be affected. The result is not a very serious loss of illuminating power. The same results will be obtained with the glass mirror, if the velocity is high and the missile small. In that case, it is probable that a small hole will be punched out and from this hole will radiate numerous cracks, the pieces of glass between these cracks retaining their position, and the illuminating power will be reduced by the area of the hole and the absorption of light by the various surfaces of the crack. It is not probable, but possible, that the whole mirror will be rendered useless by such a missile. Slow moving small missiles and large projectiles would probably completely demolish the mirror. The segmental mirror, however, would limit the extent of the cracks to the injured segment, unless a joint were hit, and at least three-fourths (in a four-segment mirror) of the mirror would remain intact. Injury by missiles is not believed to be a very serious feature, for all evidence of experimental work indicates the great difficulty of successful fire at a

search-light. The reasons for this are (a) the blinding effect when the light shines on the marksman; (b) the small target when firing from the side, or not in the line of the beam; (c) the difficulty of ranging when there are no other objects with which to compare position and distance.

EASE OF CLEANING.

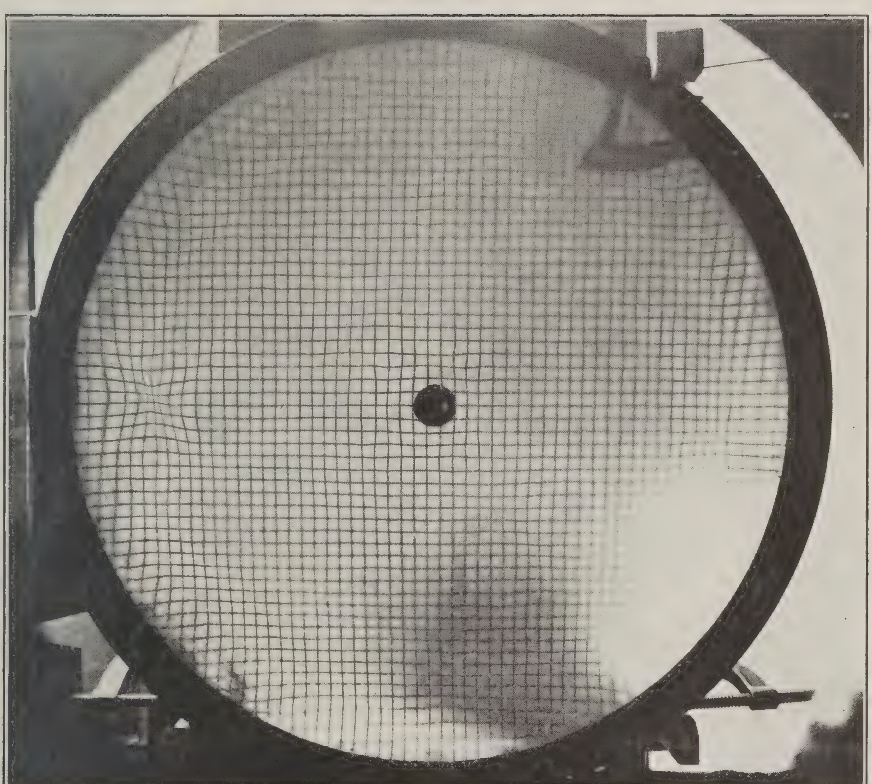
The glass mirrors require and can receive no cleaning of their reflecting surface. Dimming can occur only through dirt collecting on the front of the glass, and this dirt can be removed with soap and water, and the glass dried and polished with soft cloths and tissue paper without damage. The metal mirror, however, since the reflecting surface is the front of the mirror, is liable to damage by careless cleaning. Cleaning off of dust and dirt is not all that is required, repolishing of the surface to remove other deposits is necessary. For this there is furnished with each mirror a set of polishing materials of different grades of fineness to be used in succession, and minute directions as to cloths to be used, pressure, and direction of rubbing. The reflecting surface is easily injured, and this possibility of injury by contact is always present both in handling and cleaning, especially by untrained and awkward soldiers.

LIGHTNESS.

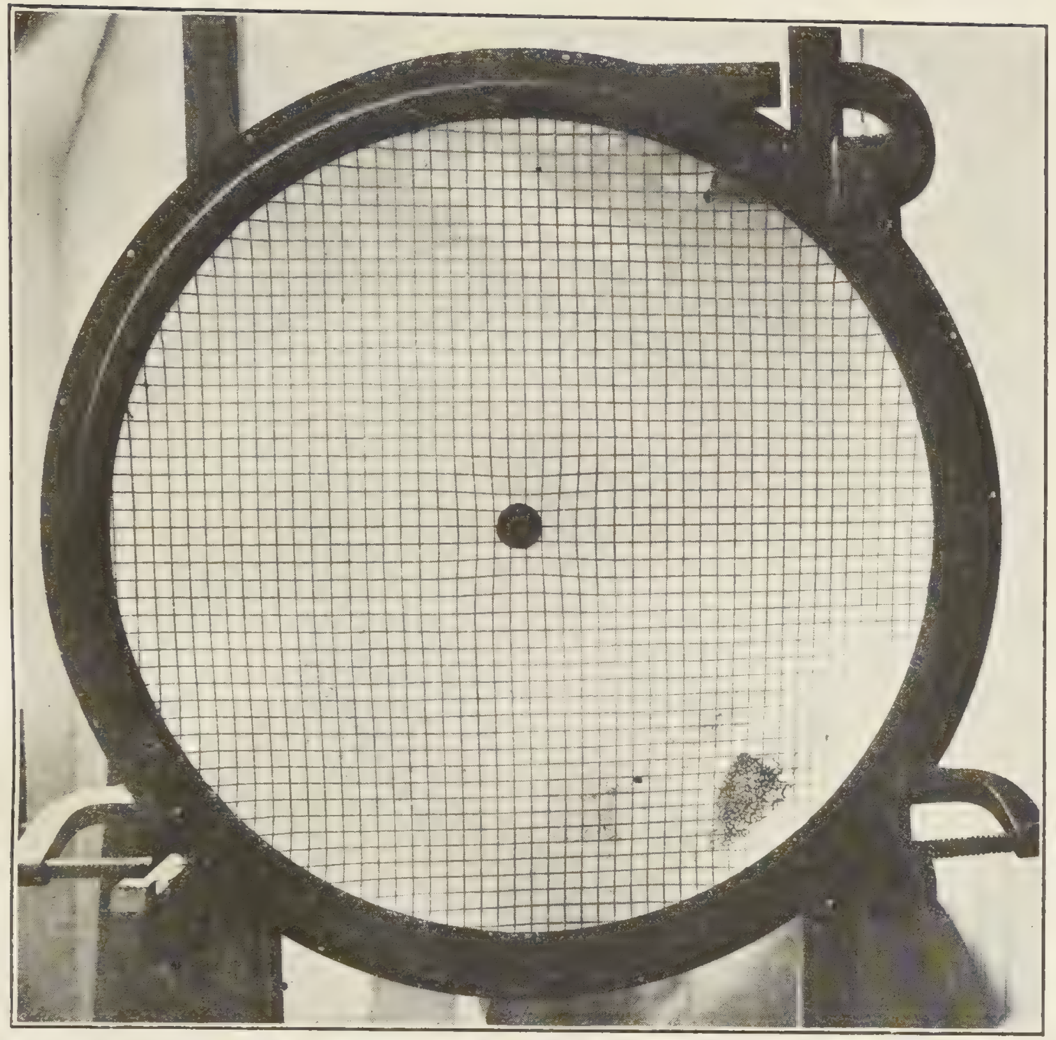
The metal mirrors are much lighter than the glass, and hence impose less strain on the mechanism during transportation, less effort and lighter weight of parts for maneuvering, and less liability to dropping and consequent damage in handling.

RÉSUMÉ.

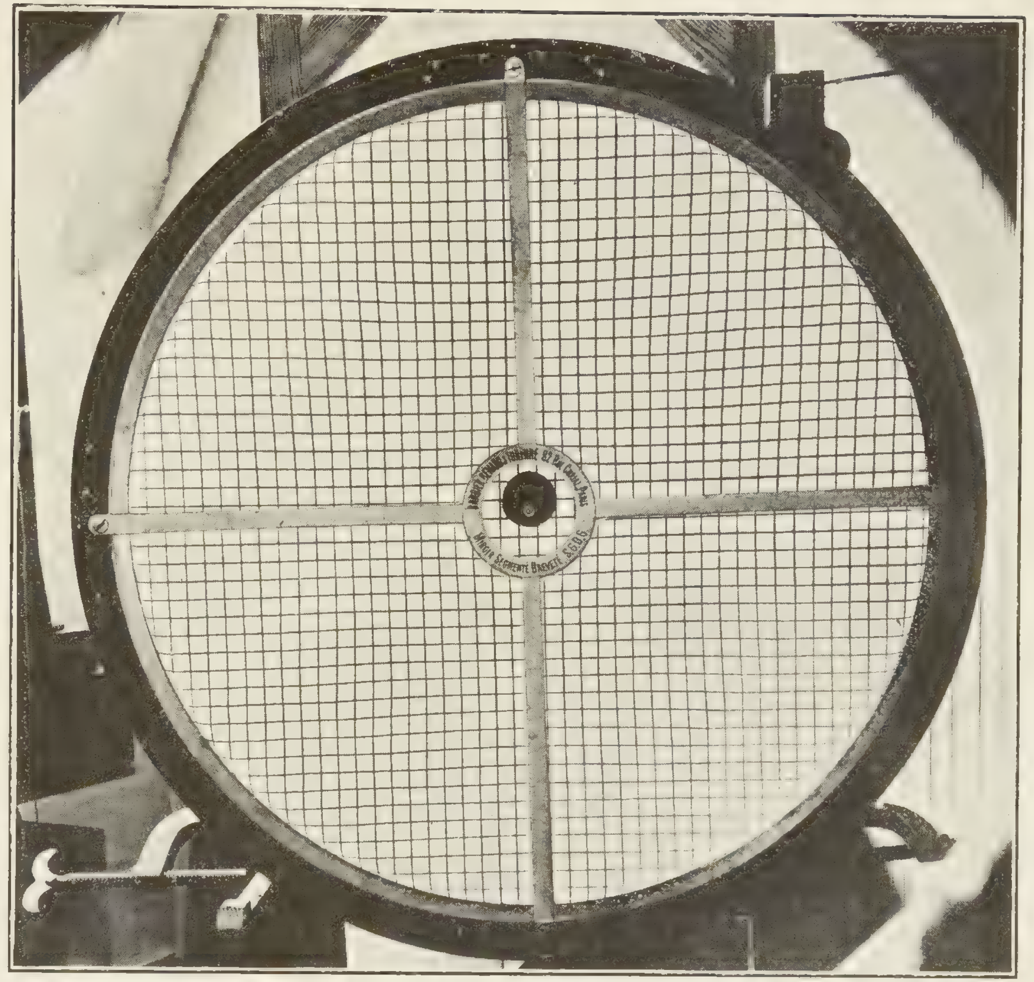
The metal mirror possesses advantages of ruggedness, immunity to damage through shock or missiles, and lightness, and is not the equal of the others in other respects. The plain glass mirror is greater in illuminating power, ease of cleaning and mechanical perfection than either of the others. The segmental mirror is but slightly inferior to the plain glass mirror in illuminating power and ease of cleaning and its equal in permanency of reflecting surface and immunity from damage in cleaning, and is decidedly superior to the metal mirror in these points. It is probably but slightly inferior to the metal mirror in ruggednesss and immunity from shock or missiles. As before stated, its weakness is its pos-



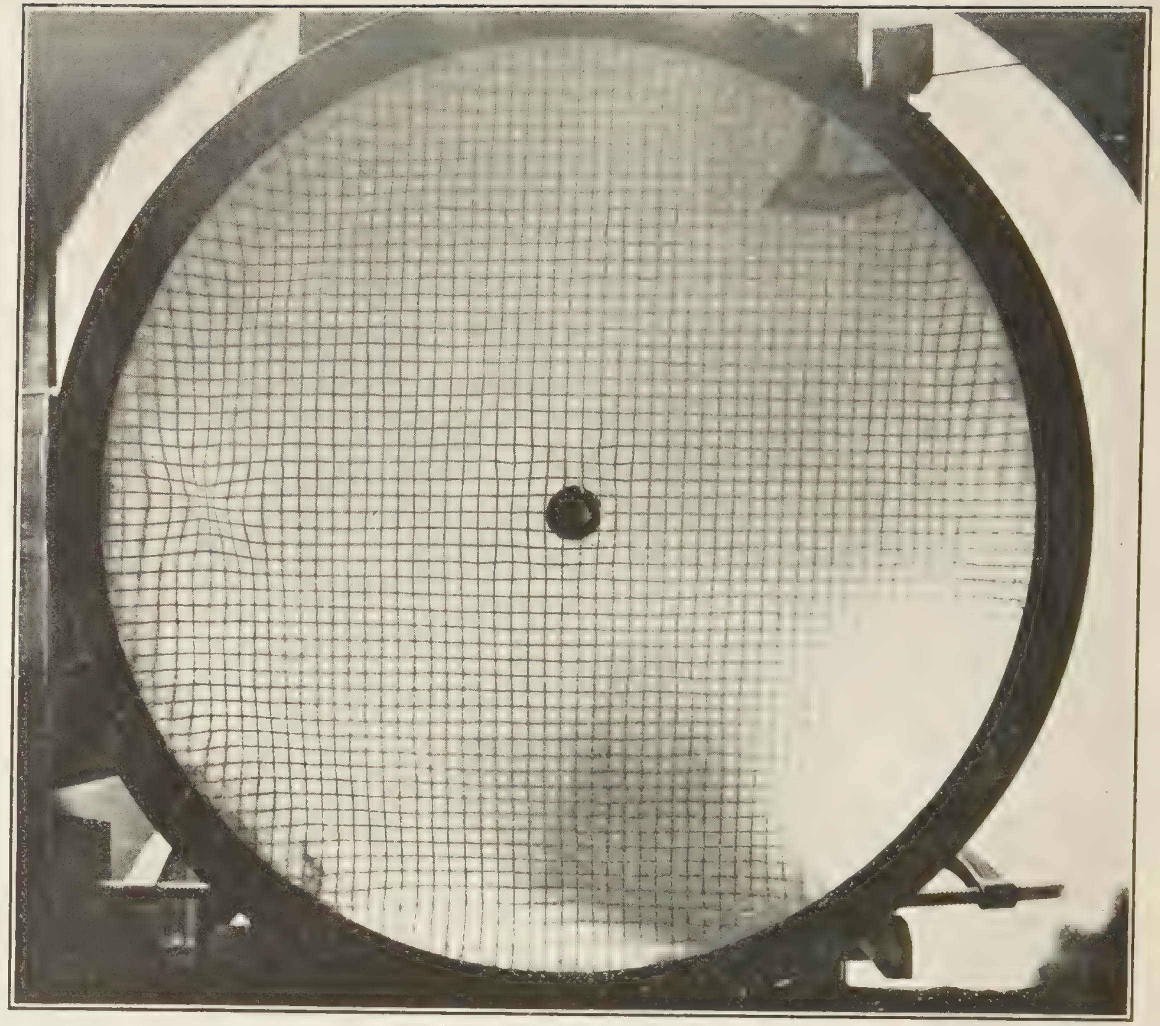
30-inch Gold Metal Mirror.



24-inch Single-piece Glass Mirror.

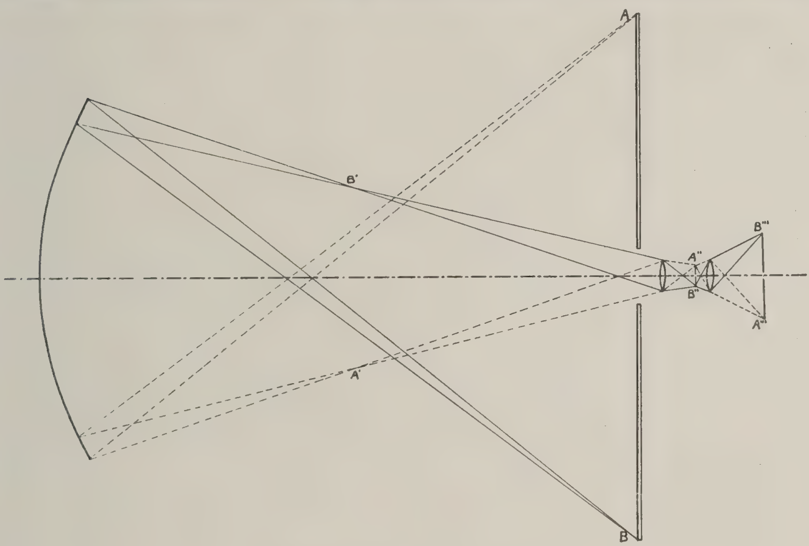


30-inch Segmental Glass Mirror.



30-inch Gold Metal Mirror.

sible departure from mechanical perfection, due to cutting and resetting or the replacement of segments. However, an absolutely perfect beam is not essential to the successful operation of field search-lights. Their function is more that of an illuminating light than a searching light and therefore absolute concentration is not necessary, and at ranges within active attack would even be a drawback, since the extent of front illuminated is very small. A more diffuse light, therefore, is, unless it passes all bounds, rather an advantage than otherwise, provided the light is uniformly distributed. The attempt to gain diffusion by throwing out of focus always results in unequal distribution. Moreover, even with



Method Used for Line Test.

an imperfect beam (if it can be called imperfect) the segmental mirror is far superior to the metal mirrors. It is not believed, therefore, that the slight lack of perfection is a serious handicap when the other possible advantages are considered. The cuts herewith are from photographs taken of the 30-inch segmental mirror, a 24-inch single-piece glass mirror, and the 30-inch gold mirror now at the School.

The photographs were made by the method indicated in the line drawing. A screen of dull white finish having black lines drawn upon it as shown in the photographs, is set up facing the mirror. Behind the screen is the camera which, through a hole in the screen, photographs the reflection of the screen from the surface

of the mirror. All that is necessary is that the center of the camera lens be upon the axis of the mirror. This method shows the mechanical perfection of the grinding and polishing, since surface inequalities will be shown by irregularity of curvature of the lines, but is not a true measure of the perfection of theoretical profile. Another method must be used to determine this. The mirror is set up facing the sun (in order to obtain parallel rays) and covered with an opaque mat applied to the surface. The mat should be formed of successive narrow rings of material. Beginning at the center or at the edge (assume the center) a ring is removed and the point where the rays come to a focus is noted. This ring is replaced and the next one removed, the focus of the zone being noted as before. This is done for all the zones of the mirror. Then, assuming a profile section and plotting to scale, the sum total of all lines drawn from the foci to their respective zones should be included in a space (at the mean of the foci) not greater than the diameter of the crater of the positive carbon which will be placed at the mean focus. The mirror is satisfactory if it fulfills this condition. The limit of error given above is allowable since the crater is not a point source of light, and hence the beam would be, even with the most perfect theoretical form of profile, slightly divergent.

The screen used was of wood, perfectly smooth and undeformed by warping, the lines were true and of uniform width and spacing.

It will be noticed that the glass mirrors are both superior to the metal mirror in

a. Definition;

b. Perfection of surface;

c. Regularity of form.

The large imperfection near one edge of the metal mirror is caused by a dent. This is not noticeable to the naked eye. The very small waves with their ridges radial can not be detected without careful observation, but are shown prominently in their effects in the photograph. The mirror is at its worst at and from half way to the edge. It is on this more imperfect portion that the maximum light from the carbon falls. The very dark and light spots were caused by reflected light from the screen and are not the fault of the mirror.

As between the segmental and single-piece mirrors there is little to choose. The imperfections of each lie at the center in the shadow of the negative carbon, and hence have little or no effect on

the beam. That the segmental mirror is so perfect, however, is but an argument for its adoption. The spots in the single-piece mirror are due to the deterioration of the silvering. Even in its present condition it is the equal of either of the metal mirrors in illuminating power, though having only 64 per cent of area of beam and using only five-eighths the current used for the larger mirror.

Appended hereto is a translation of a letter accompanying the segmental mirror on its arrival. The makers there make certain statements of fact which seem to bear out the conclusions of the writer and also make some claims which are believed to be susceptible of proof as far as our experience goes, principally with regard to the mechanical perfection of spare segments and their successful installation.

BARBIER, BENARD, AND TURENNE.

NOTE ON SEARCH-LIGHT MIRRORS.

The experiments which have been undertaken for several years on metallic mirrors have not given expected results, and another method has been sought to obtain that result; that is, by making glass mirrors less fragile. Metallic mirrors have several disadvantages; first, no matter how much care is used in working them, their surface can never be as perfect as that of a glass mirror. There are always slight irregularities which cause the beam from the metallic mirror to depart from the cylindrical form and become conical; that is to say, divergent. It follows that the intensity of the beam is diminished; therefore, the beam from a metallic mirror can never be concentrated on one point as well as can the beam from a glass mirror, for no matter what the position of the luminous crater of the light relative to the metallic mirror there is always a very marked divergence in the beam. We are able to say that in France there has been for several years the impression that metallic mirrors would give good results, and they have even gone so far as to draw specifications in which it is said "That the new armored ships shall be furnished with projectors for metallic mirrors."

Trials have not given expected results; consequently, it was later decided that the projectors should be fitted with a permanent arrangement so that metallic or glass mirrors could be installed as desired, but the metallic mirrors gave no satisfaction and the new projectors which we now have under manufacture, for example,

for the *Danton,* will be furnished only with glass mirrors. It seems to us then that the problem of the employment of metallic mirrors is far from being solved.

Second, it has been believed that the surfaces of metallic mirrors would be very easy to maintain; but it is not so. The surfaces of these mirrors are extremely delicate and it is very risky to put them into the hands of the soldiers who are charged with their maintenance. These mirrors are very easily injured, and their reflecting power diminishes very quickly. This is proven by the fact that during a fog there is deposited a film of moisture on metallic mirrors which causes their illuminating power to become insignificant. The failure of the metallic mirror having been established, as one might say, we have sought a means of perfecting the glass mirrors in a simple manner and which will give every satisfaction, and the process at which we have arrived seems to us to have solved the problem. Besides, as soon as we spoke of this scheme to the Minister of War and to the Minister of the Navy, they did not hesitate a moment to give us orders for mirrors up to 1½ meters in diameter, and indeed the Navy has given orders at all its yards that whenever it should be necessary to resilver any mirrors, to send them to our factories that, besides resilvering, we might proceed to cut up the mirrors into segments before returning them; and we receive daily not only mirrors originally of our manufacture, but mirrors from other houses which have been furnished to the Navy, both of the Mangin type and of the Parabolic. We believe that this question may be of great interest to you, and we would like to undertake this work on any mirrors which you may possess.

Further, if one possesses several old mirrors having practically the same profile, it is possible for us to make them over, using pieces of new segmental mirrors. This question is very important, especially for mirrors of large size.

Finally, to make these repairs there is no need that we have the complete mirror, but only portions of the mirrors. For example, the Minister of the Navy in ordering from us a projector of 1½ meters in diameter with a segmental mirror of six parts, has ordered that only three segments should be furnished.

At the time that we proposed this arrangement to the Minister of War the following objections were made: 1st. Will the beam from a segmental mirror be as homogeneous as that from a mirror

made from a single piece; and 2d, how would the pieces act if there should be an accident to one of the segments?

Experiments made on the first mirrors of $1\frac{1}{2}$ meter diameter furnished the Minister of War and the Minister of the Navy have confirmed what our preliminary studies have indicated; that is, that the beam from the segmental mirror is in all respects identical with that from a mirror in a single piece.

Indeed, during manufacture we do not make single pieces, but the complete mirror in a single piece, and it is only when all the work on the mirror has been finished that we proceed to cut it up. Moreover, in the tests which have been made by these Ministers, they have built up segmental mirrors with the pieces from several different mirrors, and the beam has been homogeneous. In answer to the second question, we can state the following fact. It is that, due to careless handling, a segment having been cracked, the pieces remained in their frame without moving, so that to the hand not even the slightest displacement could be perceived; and after this the department decided to continue operations with this mirror without replacing the cracked portion, since the beam from the projector was entirely satisfactory. It is now more than a year that the projector has been operating in that condition, and the mirror is still as good as a new one.

With regard to heat the mirror acts very well. We can cite as an example an order that we received from Canada for mirrors of 40 cm. in diameter before which there was an arc taking about one hundred amperes.

All the mirrors furnished by other houses failed under these conditions and broke very rapidly. We then furnished segmental mirrors in four segments and our customer has said to us that he has never had any breakage since using our product.

We may remark that the central portion of the mirror may be removed, but no ray of light falls on this part of the mirror, which is in the shadow of the negative carbon. In the French Navy they prefer to leave an opening in the center of the mirror for the following reason:

Following cannon fire, if the openings in the projector are closed the pressure on the mirror becomes quickly very great and may cause breakage of the mirror, while if an opening is left in the center of the mirror the pressure is balanced on both sides and, consequently, rupture is avoided.

IMPORTANCE AND USE OF SEARCH-LIGHT PROJECTORS.*

[From *Internationale Revue*, August, 1912.]

The search-light projector itself has now attained a high state of technical perfection and great progress has also been made in the methods for applying it. The projector may be installed at some fixed point, such as on war vessels and in permanent fortifications, etc., or it may be mobile or portable. With the portable light, it is always a question of trying to accomplish something with mirrors of only a small diameter.

The usual diameters given to mirrors are 90 cm. (35.43 in.), 60 cm. (23.62 in.), and for portable lights, 25 cm. (9.84 in.). For siege operations all three sizes may be suitable, but for field operations only the last two will serve. There are also portable search-lights in which the light is produced by chemical action (acetylene), but these are inferior in power to the electric light and their operation is much more difficult.

Vehicles † on which the projectors are transported are constructed of different form, according as to whether they are to be used in siege operations or field service. For the latter, the arrangement of a field-carriage with limber has been adopted, as we show further on.

VARIOUS USES OF PROJECTORS.

In the course of the war of 1870-1871, several projectors were used at Paris, but without any great success. The electric dynamo promised considerable importance for the employment of projectors during the war. The construction of illuminating apparatus was therefore begun, but, because of its slight mobility, it could be used only during siege operations and then only on the defensive side. Fortified places were but scantily provided with projectors, and, since it was in the first stage of development, there was but one size of mirror (90 cm.) ; it was hoped that with that size all the situations arising during a siege could be properly faced. No one occupied himself with the tactical use of projectors, and the night exercises where they played their proper part were

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*Extract of an article in the Militar-Wochenblatt, Nos. 59, 60, and 61, 1912. Translated by Lieut. William F. Endress, Corps of Engineers.

†The joint-stock company of C. D. Magirus, at Ulm on the Danube, has specialized for several years in the design and manufacture of projectors mounted on vehicles.

rare. The sites chosen for the apparatus were often just the reverse of what they should have been and but little thought was given, indeed none at all, to the use of lights according to the tactical situation. The troops were not sufficiently interested in these new engines of war and, also, they could not believe that there was any well-advised use for them.

Things changed somewhat after the introduction of rapid-fire arms. Night operations then began to take on a greater importance, and following that a more serious view was taken of the question of search-light projectors.

The Russo-Japanese War, and especially the siege of Port Arthur in 1904-1905, very greatly modified the ideas which had been held up until that time on the subject of siege operations. Fighting at short distances again played an important part, as it had done in the seventeenth and eighteenth centuries. The implements used in fighting, large and small, which had been considered as obsolete, returned to the stage, although modern science modified them somewhat. The illumination of the immediate surrounding territory assumed considerable importance and that not only in siege operations but also in the field, and the employment of search-lights for sweeping the country in field operations dates from the campaign in Manchuria. By that time the designers had practically perfected the projector itself, but equal progress had not been made in considering its proper employment and there still remains much to be done in this respect.

If the search-light were an arm, an engine by the help of which one might kill the enemy, there would be no delay in discovering the best way to utilize it. But the point is often lost that it is the search-light which may make possible the annihilation of the enemy. There are even military experts who declare that search-lights are an incumbrance and a nuisance.

Such opinions can not be based on the characteristics of the projectors themselves, for their light range and illuminating power are good. They can be due only to the methods of employing them which have been followed up until now, and for which there is no real experience; for the numerous exercises that have been gone through with up until now furnish no data of value. A projector, like a gun, gives results only when aimed and handled in a rational and well-thought-out manner. That is possible only when there is an observation service which can adapt itself to the tactical exigencies of the moment. To obtain the latter is the

fundamental reason for lighting up the terrain. If the observation service fails, the projector can not render good service and it often becomes a nuisance; but the results obtained by the observation service do not depend only on the personality of the observers nor their skill, their clear grasp of the tactical situation nor the rapidity of their decision. It is always necessary that the commanders should be able to appreciate the search-light service at its proper value and that they accord to it the attention which it deserves. The commander who neglects to keep in hand and in touch with the personnel of the search-lights and of the observation service during a tactical situation has only himself to blame if the apparatus works poorly or even becomes prejudicial.

The position of observing officer is extremely important, and it carries a very great responsibility. It is indispensable, therefore, that there should be systematic preparation for that service. These officers should not only be acquainted with the mechanical operation of their projectors, but should know equally as well how to use them to the advantage of their own troops and the disadvantage of the enemy. Only continuous practice will produce good observing officers, as well as a capable personnel for the service of the apparatus.

The many problems that the projector is suited to solve are not widely known. Drills with projectors, in the course of which only small units of troops are put in action, or where outlined troops, or even very small groups or small patrols are used, are not of any value for the large search-lights nor for the personnel of the observation service. If for siege exercises one always chooses the eternal drill ground, where the digging of trenches is always forbidden, there will be shown to the troops and to the observation personnel absolutely nothing which has anything in common with what occurs in actual war.

But conditions are especially bad, when, on an extended front, there is but an insufficient number of search-lights. The work devolving on each is then beyond its capacity, and as a result it is often concluded that the projectors are not equal to what may be expected of them.

It also happens that the functioning of the projector itself, taken alone, is considered insufficient. It is because of this that one often hears: "The range of large projectors is not enough for artillery; the lighted space is too small; those objects which remain

in shadow can not be discovered; the ranging of artillery at considerable distances is impossible because of the great contrasts between light and shadow; the search-light does not illumine properly except in complete darkness; the illuminating power is weakened by moonlight and haze, and the projector is of no value at all during fog, snow storms, etc.''

All these criticisms are founded on the light itself; but what engine of war is there which is not without its weak points; and in the future we will have no ideal lighting of the terrain any more than we will have ideal weapons. The point is that every engine of war, the search-light as well as others, must be rationally served

Fig. 1. French Automobile Search-light on the Road. (See p. 52.)

by the personnel attached to it. Only then will the defects be in the minority as compared with the advantages.

In case any doubt remains as to the preceding, let us add some explanation:

There is nothing to be said so far as the technical personnel of the projectors themselves is concerned. The groups which have received special instruction wish, almost without exception, to understand their duty, but a greater number of practical drills is necessary, combined with other arms, in order to teach them the tactical side of the use of search-lights.

For search-lights in fortified places these drills should represent the successive stages of a siege, and should resemble, as far as is

possible, actual war conditions; while for field search-lights they should resemble conditions occurring in actual combat. All these exercises should be undertaken with full-size bodies of troops, on the attacking side as well as on the defensive, and then there should be put in action the number of search-lights corresponding to the situation. The director of the maneuver should require that the commanders of the troops dedicate a special paragraph of their orders to the search-lights, in order that their personnel may know just what they are to do, and also in order that the commander may take the responsibility for the part assigned to the search-lights.

The observing officers should thoroughly know their business. In fortified places it is always possible to arrange for a nucleus of officers trained to this service. It will not be necessary, except in case of war, that these officers should be employed on other duties or in other fortified places. There is an advantage in this that the observers may follow a course of preliminary training lasting about a week, the best time for which is when the technical instruction is being given to the personnel charged with the service of the search-lights. During the course of training, there would be shown to the observing officers the methods of utilizing the projectors, and for this they should be made to see the various duties devolving on the search-lights in the simpler situations arising in war, for which the smaller units of troops will suffice. The knowledge thus acquired will be extended by holding exercises at least once each month, using the projectors.

The extent of this essay does not permit of a detailed examination into all the fundamentals to be taught observing officers, and we are therefore limited to citing some of the more important points:

1. The observing officers remain on the ground in front of the projectors as long as the enemy will permit them. They are connected by telephone with the commander and with their own light. In this way, they can first communicate to him the results of their observations and then receive from him any information he may have and take his orders. The telephone communication with the light allows the direction of the light towards desired points.

2. Whenever possible, it is necessary that the observing officers make, during the day, a careful reconnaissance of the ground which their light is called upon to illuminate.

3. They determine the actual position of the light, according to the orders or information given by the commander.

4. The lights should operate intermittently only.

On paragraph 2.—The ground presents a very different aspect, when lighted by a search-light, from that which it presents during the day. The daylight illumines all the folds in the ground, which is not the case with light from a projector, because the source of

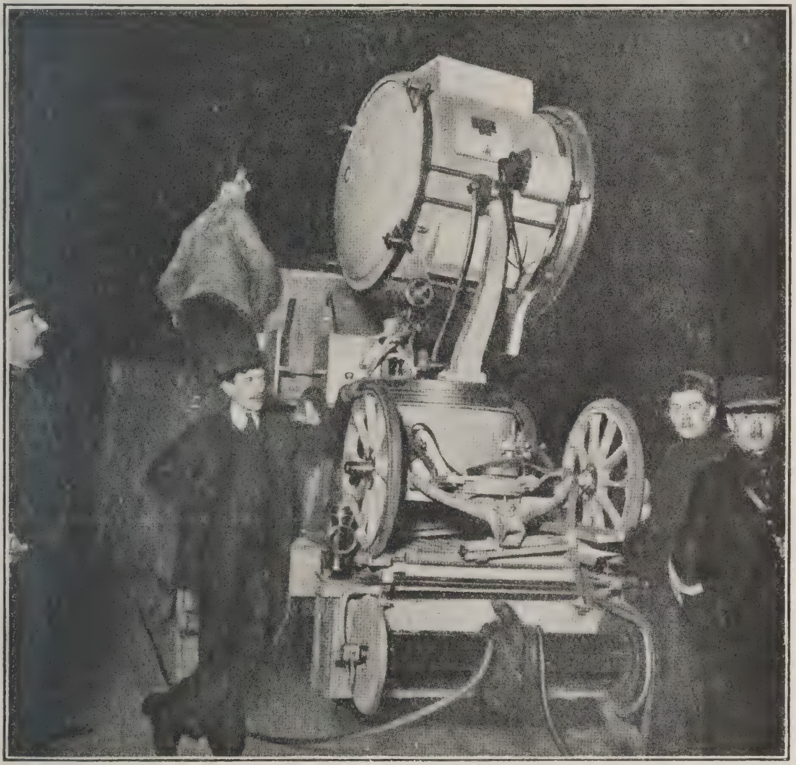


Fig. 2. French Automobile Search-light Under Operation. (See p. 52.)

light is at but a relatively small height above the surface of the surrounding country. This difference is especially marked in flat country, but not as much so as when the ground contains trees, hedges, etc. In the latter case, long shadows are produced which contrast very strongly with the brilliant light. Means are therefore sought of placing the light at a certain height above the ground. The older apparatus consisted of a telescoping tower of wood, while the modern consists of two tubular steel telescoping

masts to which the projector is attached. In rolling or mountainous country it is possible by choosing an elevated site to relieve somewhat, at least for the immediate foreground, the inconveniences due to the shadows. The electric light is also the reason that the colors in the country illumined by the projectors appear to be different than they are by day. Also, those objects which are directly in the full light appear to be very much larger than by daylight. In addition, the intermittent light confuses the observer and prevents him from recognizing objects with the same facility as if the lights were continuous. Consequently, it will be easily understood that one needs a very well trained eye in order to observe rapidly and with certainty.

On paragraph 3.—The commander determines on which portion of the offensive or defensive front the search-lights are to be established. The observing officers then seek out and decide on the exact situation for the projector. The dominant points of the terrain are the most suitable; they allow a flanking light, thanks to which it is often possible to light up folds in the ground surface which run more or less parallel to the front of the position. In choosing the site for the projector it must be remembered that cover for the field carriages is necessary, and those sites must be avoided to which access is difficult.

On paragraph 4.—The search-lights should discover the enemy and allow of fire against him by the various arms. In order to accomplish this, it is necessary that they should be kept intact for the commander for as long a time as possible. The adversary will certainly attempt to suppress this illumination which inconveniences him so greatly. The continuous light will draw the fire of the enemy's artillery and infantry, and although it is extraordinarily difficult to destroy a search-light in this way, it can be accomplished in time. The intermittent light, on the contrary, very greatly troubles the enemy who tries to get the range by firing on the projector. It may therefore be conceived that in some measure the light may really become an element of defense.

The intermittent light has still another advantage. The enemy can conceal himself from such a light only with great difficulty. At first it was the custom to follow the ground slowly with the light, making the cone of illumination pass slowly from right to left and inversely. In order to conceal themselves from this light the troops were ordered to lie down and not to move as soon as the luminous cone approached, and it would then pass over them

without the observers being able to ascertain their presence. But if the light illumines only at intervals, and now here and now there, the enemy is surprised while he is in motion. It does no good to lie down after that, if the observers are on the alert and well trained.

It is evidently necessary, for all this, that the light with its shutter closed, should be capable of being pointed towards any predetermined position on the terrain. The appliances necessary for this are incorporated in the apparatus. If the observers prepare, while it is daylight, a simple sketch of the terrain, made from the future site of the projector, and on which they have marked the prominent points of the ground, they will be able, after operating for a while with the intermittent light, to place these points in agreement with the azimuth circle which is carried on the projector carriage.

We have seen above that, among the things which should exist permanently in a fortified place, is a small body of observing officers taken from the troops and who have received a thorough course of instruction. It is otherwise for the field armies. The light 60 cm. wheeled projectors, which exist as a part of the army corps, will be attached to different units according as circumstances may arise, and these units ought to furnish the observing officers. But these officers will find themselves charged with duties absolutely new to them. This fault will be obviated to some extent when the employment of projectors becomes more a matter of common knowledge, following night exercises which can be carried out on the drill grounds, or in the neighborhood of the garrisons; and when in their review of the operations, the directors of the maneuver shall accustom themselves to examine into the action of the projectors for the benefit of the commanders and the participating troops.

DIVERS FORMS OF PROJECTORS.

Up to this point we have spoken only in a general way of the employment of search-lights, and especially of the heavy projectors of great range (90 cm.) and of the light ones (60 cm.). But as, in order to arrive at some desired end, pieces of different caliber are necessary for the artillery, just so there must be projectors of different kinds.

The first question arising is, Is there any need, in a war on land, of search-lights of greater range than that of the 90 cm. light? Except for very special cases, the answer is "*No.*" There never

exists in the country surrounding fortresses a terrain which is absolutely flat for several kilometers, and which contains neither fields nor buildings. Now, the farther the portion of ground to be illuminated is from the site of the projector, the flatter is the angle between the luminous cone and the surface of the earth, and the greater are the shadows cast by the least inequalities of the ground and by the smallest vegetation, even very low, such as hedges for example. These shaded parts of the terrain indicate at once to the enemy the points where not only is he protected against the light of the projectors, but also from view during the day, unless he is observed from high in the air. It will not be difficult in that case to advance the infantry and artillery under cover, even for considerable distances, and even to allow the latter to go into battery under cover.

Because of the very great distances at which the operations of a siege are developed, a search-light, no matter what its power, can never pick up the points where batteries have been constructed, provided the sites have been carefully chosen. In such a case there is only one method to be followed—that is to make a sortie, or at least to depend on balloons, aeroplanes, etc. There is hardly any excuse for projectors of great range, for they may even become a nuisance, in that they may reveal to the enemy certain of our own dispositions.

They are of no greater value in the artillery duel, for the artillery never establishes itself in the open, and observation at long distance with the alternation of brilliant light and black obscurity produces little in the way of results. The 90 cm. searchlights are effective to 3 kilometers under favorable conditions. This is, in general, the distance at which, in siege operations, the infantry begins its forward movement against the sector to be attacked. From the beginning of that movement the lights will sometimes be able to discover the enemy, though even then it will often be difficult. And it would never be possible unless the observing officers are well in front.

The true field of action of the search-light is under 2 kilometers and the attacking infantry will, therefore, usually be easily discovered by the 90 cm. projector. But, the nearer the attacking infantry approaches, the greater is the possibility that large projectors will be put out of service one by one, although this is difficult to do.

At very close quarters the usefulness of large projectors is

lessened. The frontal angle to be covered by a single projector becomes so great that it will have to be turned very much to one side. That is not only a disadvantage, because a luminous cone directed thus to the side always masks something, but it is also very dangerous to the friendly infantry both in their firing trenches and in the offensive operations which are attempted by them.

The last phase of close-quarter fighting is composed of a series of isolated actions and enterprises, undertaken by small units, sometimes the smallest organized fractions. By working energetically until a favorable opportunity presents itself, the most valuable results are possible. The suddenness of these enterprises very often prevents any notification to neighboring troops and more often to the large projectors, which, installed sometimes far from the place where the action develops and placed under the orders of other commanders, may, without wishing to, compromise and even annul the success of the enterprise by lighting up a predetermined point at the wrong time.

During this close-quarter combat, all superfluous light should be avoided; and it is a fact that large projectors give at such times an unnecessary amount of light. The expedient of retiring them, as is done with the heavy artillery, in order that they may work from a position farther in rear is not applicable here, because the slight distance which separates the two adversaries would cause them to light up both positions at once. At this stage of the action the large projectors have come to the end of their usefulness in lighting up the surrounding battle; the great handicap that they may from then on become to the friendly troops requires that they confine themselves to lighting up only the more distant field of attack, in order to discover the troops, transport of material for the assault, etc., which may be crossing the exposed places.

In order to illumine the field of close combat, it is necessary to have recourse to the small projectors which the troops can use as most convenient to them. The sector of ground assigned to each little projector must necessarily be small in order to avoid obliquely directed beams. The range of the projectors for this service should not be very great. In the fight on the immediate foreground projectors having a range of 300 meters are sufficient. But, since we must consider that in the course of the close-range fighting the large projectors may be put out of service, and that then their work must be done by the smaller lights which are sometimes at considerable distances from each other relatively, it would be

better to choose lights of 800-900 meters range. This requirement is filled by the electric projector with a mirror 25 cm. in diameter. As auxiliaries, there would be, especially in the interior of permanent works and for special purposes, some projectors a little more powerful, say about 35 cm. (13.78 in.), for lighting up long lines of obstacles, for example, or to allow the covering of intervals.

Search-lights having a diameter of 60 cm. which, in general, are designed for field service only, can be used just as well in sieges as light-weight projectors. They have the great advantages of mobility and can, up to a certain point, take the place of the 90 cm. lights. Their range is about 2 kilometers under favorable conditions.

Besides search-lights, there are also available for lighting up the ground over which the attack moves, fire-balls, fusees, torches, etc.

The fighting for fortified positions in field warfare presents situations analogous to those of siege operations, except that the successive stages develop more quickly because the fortifications are weaker; the cannon which the artillery puts in action are lighter and possess greater mobility; the projectors to be used in position combat should also possess greater mobility.

The light 60 cm. projectors fulfill these requirements. The carriage, which in appearance is like the artillery field carriage, is able to travel rapidly over the country away from roads in order to get quickly to its position and commence operations.

Besides these projectors, small projectors are necessary to light up the immediate foreground, just as in siege operations. For transportation, the projectors are grouped on a special carriage which should possess the same qualities of mobility as does the 60 cm. carriage.

Finally, the service of the projectors should be organized as a distinct entity, and the whole placed under a single commander.

Usually, horse-drawn projector carriages are preferable to automobile carriages. The light-weight projectors of the field armies should be able to traverse all kinds of country, just as the field artillery does. With a team of six horses this desideratum is possible, while at the present time, at least, it is not possible with automobiles. Also, it must not be overlooked that a team is easily replaced, in whole or in part, and it can also be increased in size if there is occasion for it. With automobiles the slightest trouble with the motor renders the utilization of the projector extremely uncertain.

In siege operations, especially on the defensive, conditions are just the reverse, for there is always a very complete network of military and other roads and the projectors will only in very exceptional circumstances encounter bad roads or find it necessary to cross the fields. Under these conditions, it is better to use automobiles. However, the chances always remain that the motor will be hung up on stretches requiring too great power, and one or another of the projectors may still be seen standing idle on the road.

THE NUMBER OF PROJECTORS.

In the proper use of search-lights numbers play a very important part. We have already pointed out that, indeed, there are very vague ideas as to the number of lights necessary for any predetermined situation. The newspapers have demanded, during the last few years, that there shall be a very considerable number of projectors. These demands have been very generally based on the occurrences of the siege of Port Arthur and the campaign in Manchuria, and the result is a very large number. But here, also, it is necessary to content one's self with what is absolutely necessary, for it will never be possible to get all that it may be desirable to have.

The defender needs more search-lights than the assailant. The first-named will try to explore, to show up all the ground in front, while the second will seek obscurity in which to execute his maneuvers. It is, consequently, much more difficult to make use of search-lights in the attack than it is in the defense.

I. NUMBER FOR SIEGE OPERATIONS.

It must be acknowledged that the sector of attack on a place fortified with detached works will but very rarely exceed 6 kilometers in extent. It matters little, from our point of view, whether the attacked front presents an uninterrupted line or whether there are two separate attacked points at different points of the front.

a. In the Defense.

The least number of heavy projectors demanded is one outfit per kilometer. And it should be understood that these pieces of apparatus should not be used from the permanent works, but outside of them; that is, they should be mobile. As the range of 90 cm. lights is about 3 kilometers under favorable conditions, the 1 kilometer of actual front assigned each projector will in reality be about double that at the limiting range of the light; in other

words, each light will be capable of searching about 2 kilometers of front at its extreme range. With the above allowance of large projectors it is possible to group them in twos, in order that they may aid each other in exploration and in continuing to illumine some object which should be kept visible all the time. The command of these two projectors, between which there will be such interval as the terrain demands, is naturally confided to a single officer.

There will be necessary, then, six large projectors (with 90 cm. mirrors) as well as a reserve of two projectors. The large projectors may be replaced by smaller ones (60 cm.) in many parts of the front; for instance, where woods in the foreground limit the range. The number of portable lights intended for illuminating the intervals and ditches should be made the subject of a special calculation. As for those fortresses which comprise groups of detached works, distant and near, the calculation of the required number of projectors of the various kinds (heavy, light, and portable) is made according to the preceding discussion.

On those fronts which are not to be attacked there should also be a certain allowance of projectors, but the number may be much less than is required for the sector undergoing attack.

b. In the Attack.

The besieger will certainly carry projectors along with his siege material. The allowance of the various kinds of projectors will be according to local conditions. In each engineer park there should be two to four projectors, the heavy and the light, and horse-drawn.

II. NUMBER IN FIELD ARMIES.

Search-lights find here hardly any field of activity except during combats of position, and then their method of employment approaches very nearly to that required for siege operations. The number of lighter projectors (60 cm.) which are added to an army corps is calculated just as it is for the defensive side in siege operations. There is needed, therefore, one piece of apparatus for each kilometer of front. Since, for one army corps operating alone one can count on a front of about 7.5 kilometers, eight projectors will be required, which are assigned half to each division, and these four divisional pieces are organized into two sections.

The vehicles are made up of limber and carriage. The first carries the machine which produces the electric light, while the

second carries the projector mounted on telescoping steel tubes which can be lowered to a horizontal position on the carriage during the march.

Because of the relatively short time that a fight for a position will last, it is probable that the lighter projectors (60 cm.) will remain, for the greater part, in illumination until the end of the action, and consequently the number of portable projectors need not be very great.

The increase in the wagon trains of an infantry division which results from the addition of search-lights is certainly a bad feature,

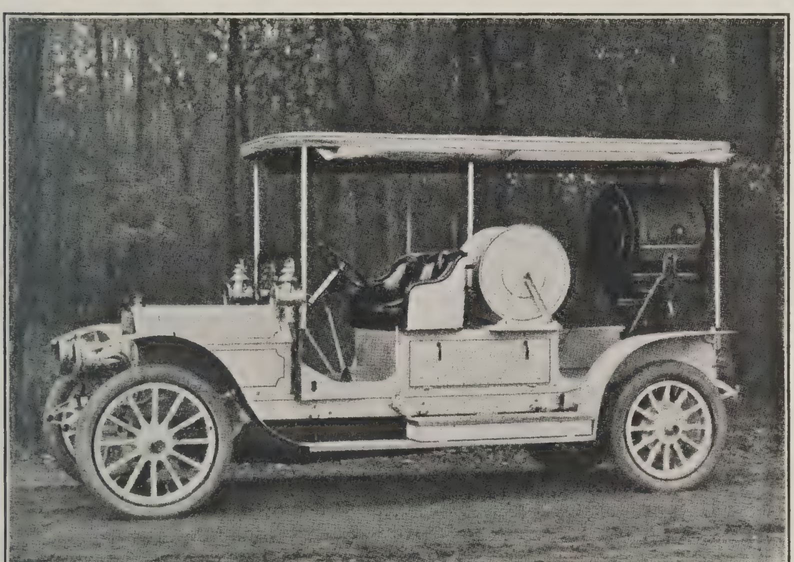


Fig. 3. German Auto-Searchlight with G 60 Projector. Siemens-Schuckert. (See p. 53.)

but it must be submitted to if preparation is to be made for all eventualities arising during a campaign in the field.

PORTABLE SEARCH-LIGHTS.*

[By Lieutenant Pölster, 1st Foot-Artillery Regiment of Linger (E. Prussia)].

The Russo-Japanese War, in which local results were largely determined by darkness, fog, and the failure of the adversaries to take advantage of these conditions, has brought to the front, in

*Translated from the Kriegstechnische Zeitschrift, No. 8, 1912, by Lieut. William F. Endress, Corps of Engineers.

our Army, the importance of thorough training of the soldier in night-work as soon as he enters the service as a recruit.

In night operations well trained bodies of troops can surely accomplish great things, even though they suffer considerable loss in the subsequent retreat, by proceeding silently and safely in operations on the enemy's flanks, surprising possible hostile patrols, and attacking the enemy resolutely and quickly without the enemy's being conscious of their presence and without allowing time for effectual protective measures. Moreover, darkness is taken advantage of as far as possible for the advance of the larger bodies of troops, flank movements, emplacing artillery, construction of cover, laying down tramways, bringing up munitions of war, etc., and especially is night training of importance now that, in modern war, there is the great opportunity for aeronautical reconnaissance and the possibility of aerial offensive measures, the value of which, in the daytime, is acknowledged by all the powers.

To what art or science are we to turn in order to prevent or expose these extensive undertakings? Here, again, technical and tactical skill must work hand in hand, and the great utility of the search-light makes this possible. Especially in Austria, Russia, Japan, and France a great deal has been done during the last year along this line, and the attaching of search-lights to the infantry regiments is prepared for or already accomplished.

As a starting point, from the experiences of the Russo-Japanese War, in which both Russia and Japan made the widest use of search-lights, not only in the field but at Port Arthur, there was established in Russia a commission to investigate the applications of search-lights with troops; and they proposed that all arms of the service should be provided with search-lights upon the outbreak of war; they also prescribed that each army corps staff and division should be equipped with a 75 cm. light, each infantry regiment and separate battalion with a 35 cm. light, and each artillery brigade and cavalry division with a 75 cm. light—all to have the same mobility as the field gun. At the same time the organization of a special body of troops for this service was undertaken, and the necessary and proper regulations were drawn up.

In France, extensive experiments have been recently made with automobile search-lights (fig. 1, p. 41, and 2, p.43) to determine their range of usefulness, and a single experimental unit was purchased. The wagon is by Dion-Bouton, the search-light by Breguet. The 90 cm. light is mounted on the automobile, which is about 18

horsepower, has a speed of 30 kilometers an hour, and can negotiate grades of about 15 per cent. The motor can be used as generating apparatus for the projector either while travelling or standing still.

In Germany, where several search-light detachments have been organized among the pioneer battalions, the units are from the firms of Siemens-Schuckert, A. E. G. and Benz, and the non-automobile units from C. D. Magirus; these latter have been especially suc-

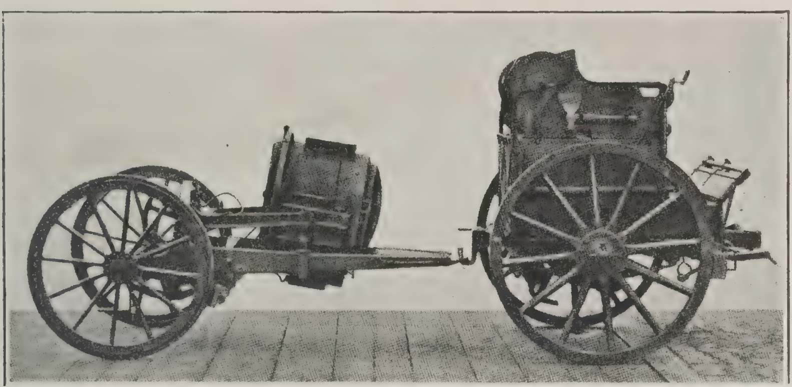


Fig. 4. German G 60 Outfit. (See p. 54.)

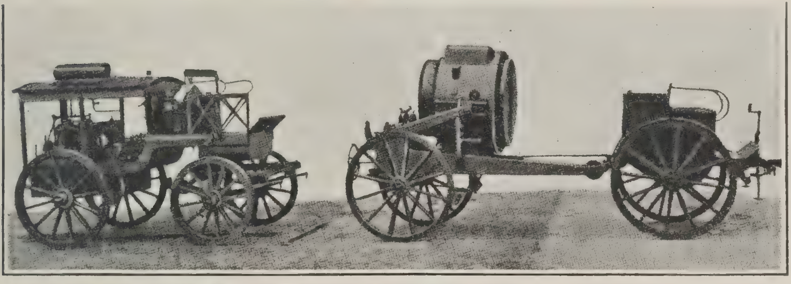


Fig. 5. German G 90 Outfit. (See p. 55.)

cessful. Fig. 3 (p. 51) shows an automobile search-light wagon with G 60 projector* mounted on a tripod. The search-light and other apparatus is mounted on a chassis which is driven by a benzine-motor of about 28 horsepower, the motor also serving to drive the 80 volt, 60 ampere dynamo at 1100 revolutions per minute. The motor is provided with a governor and double-ignition system. The radiator is designed to allow of operating the light continuously for two hours without renewing the water. The dynamo is

*Is this "Type G, 60 cm."?—W. F. E.

driven through a system of auxiliary gears which can be thrown in or out of mesh by a second lever conveniently placed on the side and similar to the speed-change lever. The ordinary fuse box for the dynamo is expanded into a switchboard box and contains the following apparatus:

1 combination volt-ammeter;
1 field regulating rheostat;
1 circuit-breaker;
1 hand lamp and protective fuses.

Above the switchbox of the generator is the cable drum carrying 150 meters of two conductor stranded braid-covered cable.

The G 60 light is easily dismounted from its rubber springs and weighs, with its tripod, 210 kilograms.

The body is designed for three seats and has canvas side-curtains.

The automobile has four forward speeds and reverse, and can take 18 per cent grades. The maximum speed is about 55 kilometers per hour, and the total weight 2,300 kilograms.

The light horse-drawn portable before-mentioned (fig. 4, p. 53) consists of two 2-wheeled units: one, the limber, carries the power supply apparatus; and the other, the carriage, carries the projector.

The limber forms a unit which comprises the 8-10 horsepower benzine-motor and a 65 volt, 60 ampere shunt-wound generator. The motor is water-cooled and is provided with a circulating pump and ventilating fan. To maintain constant speed there is a centrifugal governor, and the ignition system is double. The radiator is on the left side of the limber and on the right is the switchboard, whose equipment consists of field-regulating rheostat, measuring instruments and switching apparatus, as well as the necessary fuses and hand lamps. An explosion-proof container is mounted under the seat and contains benzine and oil sufficient for about fifteen hours operation. Space is provided under the footboard for a field-telephone set. Behind the seat is the cable-drum for 150 meters of double conductor cable of 16 $mm.^2$ section. The limber weighs, including supplies, tools and spare parts, about 1,100 kilograms.

The carriage for the projector is composed of a pressed-metal frame, on which the projector is raised from the travelling to the operating position by means of a worm-gear. The projector trunnions are supported at the ends of a U-shaped member, which allows of swinging the beam in the vertical plane. The lower por-

tion of the U-shaped piece forms a turntable, and the light can thus be swung in all directions. In the travelling position the projector is supported on rubber cushions or buffers. The projector-drum is provided with an iris shutter and signalling door of the venetian blind type. The projector, its trunnion-arms and turntable can easily be dismounted from the carriage and set up somewhere else.

This apparatus is especially suitable, because of its extreme lightness, for service in mountainous regions. It weighs only 460 kilograms, and is, because of its large wheels, easy to move about. Also, the carriage can be removed when the road is no longer pass-

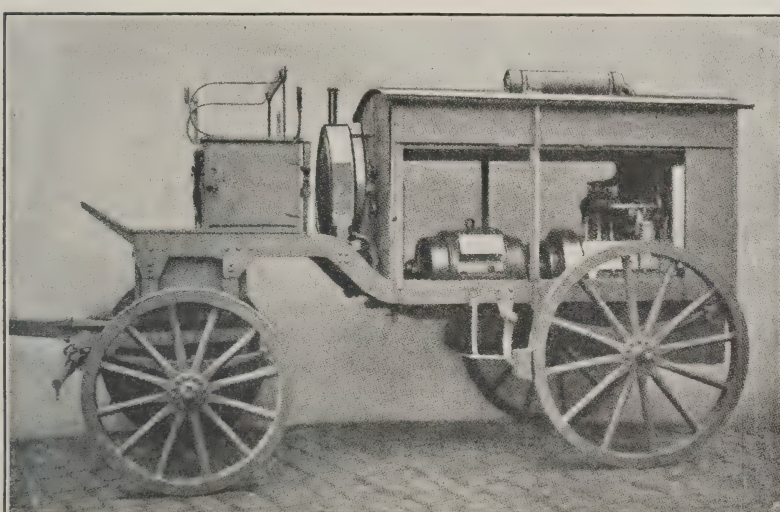


Fig. 6. German G 90 Power Unit.

able, and a body of men drag it up the steep mountain paths while the limber remains below under cover.

A later type of train with a G 90 light, suitable for the equipment of fortifications, is shown in fig. 5 (p. 53). It contains a four-wheeled power unit (fig. 6) equipped with a benzine motor and direct-driven generator of the previously mentioned type. The remainder of the power-unit equipment is similar to that for the lighter sets, except that the cable drum is left off. The weight, loaded, is 2,000 kilograms.

The cable-limber is designed for animal traction, and carries 250 meters of 2-conductor cable of 25 mm.² section. The drum is actuated by two hand cranks, which can be inserted between the

spokes of the wheels. The weight of the limber and cable is about 800 kilograms.

The carriage for the G 90 light is similar to that for the lighter set, but all parts are, conformable to the greater weight of the projector, made heavier. The weight is about 615 kilograms.

The G 90 light is easily operated; it has a parabolic glass mirror 90 cm. in diameter, and a glass front door, is provided with an iris shutter and signalling shutter of venetian blind type. The drum has hoods and cowls and openings for ventilation, and an apparatus for observing the arc. There are means for adjusting the carbons from the outside, carbon supporting arms, and a lamp taking 90 amperes at 53 volts. The drum can be manipulated horizontally and vertically directly by hand or indirectly through gears and couplings. The weight of the search-light alone is 258 kilograms, and that of the carriage with its search-light, 900 kilograms. The entire weight of the two units, ready for the road, is 1,700 kilograms.

It is not enough that we should possess or have in course of preparation a number of search-light sets in good working order for any mobilization, the principal thing is: That the troops be trained to familiarity with the features of search-light illumination, both of their own and of any hostile lights, for a horse or a man can easily betray an entire position and draw a destructive fire. For that reason the distribution of lights to all arms of the service is necessary, in order that at the outbreak of war we may be prepared and not have to undergo any surprises.

Methods and Cost of Damming the Hymelia Crevasse in the Mississippi River*

The Hymelia Crevasse, caused by the floods of 1912 on the Mississippi River, was one of the most serious breaks along the river. The break in the levee occurred on May 14. The Government attempted to repair it shortly afterward, but a second flood wiped out about $60,000 worth of work and compelled the Government engineers to abandon the task. Later, the State secured funds and undertook the work and accomplished the closing of the levee.

Hymelia Plantation is situated on the west bank of the Mississippi River, 35 miles above New Orleans. At this point the levee is 15 feet high, has a base of 105 feet and a crown of 15 feet. People living along the river became alarmed by the high water early in the spring and organized detachments to watch the levee night and day. Prompt work by these men saved the levees in many instances. Nine years ago a break had occurred at Hymelia that had caused thousands of dollars' loss, so this section was particularly feared and a strict watch was kept by the planters.

Some days before the break occurred "mud boils" were noticed in front of the levee, an indication that the water was eating away the softer earth, and an attempt was made to stop the damage being done by loading the ground over the supposed hole with sand bags. The object of this loading of the levee was to crush in the earth around the hole. Such holes generally extend horizontally into the levee. This was, however, ineffective. Early in the evening about 200 square feet of the road in the rear of the levee turned over, as though a cellar door had been opened, and the levee began to crumble away. In a week 4,700 feet of levee were gone.

After a conference with the Lafourche Basin Levee District officials it was decided that an attempt should be made to close the break, under the direction of Capt. C. O. Sherrill, U. S. Engineer Corps. The levee district was to share the cost with the National Government and placed $100,000 at Captain Sherrill's disposal. Supplies and men were rushed to the scene and work started in the shortest possible time. The method used was to drive 4 by 4 inch timber in several rows, commencing from each end of the break, filling in between the rows with sand bags as the work progressed. At one time it was estimated that there were 1,200

*Reprinted, by permission, from *Engineering & Contracting* for September 18, 1912, with two original discussions on the advisability of such closures.

men engaged on the work, filling sand bags, driving the piling, etc. All piling was driven by hand, using a small home-made pile driver. After spending approximately $60,000, the attempt was abandoned. At the time this work was going on the river stood at about 40 feet, Cairo Datum. Time after time the fierce current tore away the work and made another start necessary.

After a conference with the Board of State Engineers, the Commissioners of the Lafourche Basin Levee District decided that no further attempt should be made to close Hymelia Crevasse until

Fig. 1. View of Timber Dam Constructed to Stop the Hymelia Crevasse on the Mississippi River.

the river went down to somewhere near the normal stage. This came about the beginning of July, and the Board of State Engineers decided to start the work. The flow of water had at that time dug for itself a channel 1,100 feet wide and 18 feet deep, the river being at 32.2 feet, Cairo Datum. Mean Gulf Level corresponds to a reading of 21.56, Cairo Datum, which would mean a head against the dam of 10.64 feet. In addition to this head, there would be the head due to the current which, at that time, was approximately 5 miles per hour.

Plans were drawn for a timber dam and bids were called for, to

be opened July 10. The structure was calculated to withstand the combined heads and provided for two rows of piling having a penetration of 25 feet, 10 feet apart, with the bents on 5-foot centers. Along the front row was to be a 10 by 12 inch waling bolted to the piling. Bolted to this waling was a row of 9-inch Wakefield sheet-piling driven with a penetration of 20 feet. Where required by the depth of the channel, a third row of round piling 10 feet back of the second row was to be driven. Each bent was cross and tie braced with 8 by 10 inch timbers bolted to the piling. The work was to be commenced on or before July 20 and completed on or before August 10. A bonus of $200 a day for each day that the

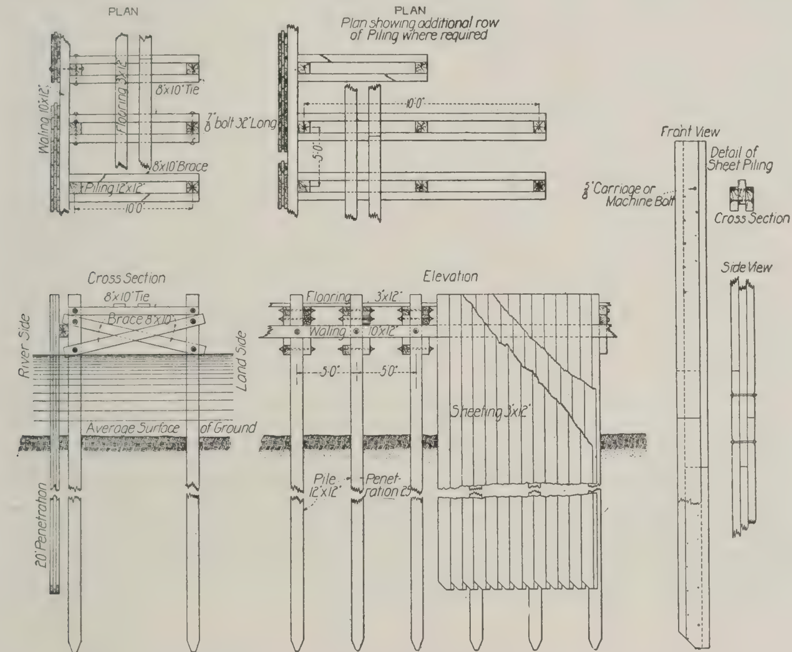


Fig. 2. Details of Timber Dam at Hymelia Crevasse, Louisiana.

work was completed ahead of time and a forfeit of $100 a day for each day after August 10 that the work remained uncompleted, was provided.

Bids were submitted by the Hercules Co., New Orleans, $55 M. B. M.; W. M. Wren, New Orleans, $75 M. B. M., and Doullut & Williams, New Orleans, who offered to do the work for a commission of 25 per cent on the cost of the labor and materials. After careful consideration and on the recommendation of Frank M. Kerr, Chief State Engineer, the contract was awarded to Doullut & Williams. The contract provided that the contractors were to furnish pile drivers, quarter boats, barges, etc., and to charge rent for them at the rate of $15 per day for pile drivers, and $10 per day for quarter boats and barges.

Immediately on the award of the contract, prices on the material were asked for from the different supply firms, and it was found that the piling and lumber could not be supplied without considerable delay unless a premium was offered. By paying an advance on the market price, an order of piling that had been sold for export and was on the wharf ready to be put on shipboard was secured. For quick delivery an advance had to be paid over the market price on practically all of the lumber that went into the dam. All of the piling lumber and hardware for the dam and all of the contractor's plant, consisting of four pile-drivers, air compressor, blacksmith shop, tools, camp outfit, and men had to be loaded on barges and towed to the site. The pile-drivers were assembled on barges; one driver was a track driver, one turntable driver and two scow drivers. Five days after the contract was signed, all of the plant needed for the work and a big proportion of the material was landed at Hymelia. The first work was to provide a kitchen and dining room for the men. This was a frame building, 125 by 24 feet, consisting of two dining rooms—one for white and one for colored—and a kitchen with an earth floor. Arrangements had been made beforehand for sleeping quarters by having a number of houses of the plantation hands cleaned out and whitewashed. These houses were situated 1,500 feet from the work. Cots and bedding were supplied by the contractors. During the first three days heavy rains retarded the work considerably, so that it was not until July 20 that the first piling was driven.

After the contractors arrived at Hymelia a change in the plans was made by the engineers, who decided that it would be better to drive the third row of piling all the way across the channel with 25-foot centers. It was also found that the bottom was composed of river sand so soft that the 25-foot penetration was secured in some cases by as few as seven blows of a No. 2 Vulcan steam hammer. On this showing, J. W. Monget, State Engineer in Charge, directed that the round piling should be so driven as to secure a penetration of 35 feet.

Arrangements for driving piling were as follows: A driver was started from each side of the channel driving the second, or inner row, dropping back of this row every 25 feet to drive a pile in the third row; then pulling back and continuing on the second row. As these drivers moved toward the center of the channel the other two drivers were started from each side driving the front row. This arrangement was continued until the center was reached, when one driver completed both rows. The drivers that were released as the center was reached were pulled back to the shore ends of the dam and put to work lining piles for the bracing. Hanging the braces was commenced as soon as the drivers had finished driving round piling in the center of the dam. These drivers followed those that were lining piles and hung the braces. A barge, containing the air compressor outfit, followed the crew hanging the braces and bored the holes for the bolts. As soon as the lining

was completed, the two drivers from that work were again put to work hanging the waling to form the shore ends. When the two drivers that had been hanging braces met at the center they were placed at the shore ends again and started driving the Wakefield sheet-piling. All of the pile-driver men were union labor. The best record for a day's work—thirteen hours—on this job was made by the track driver, which drove 93 piles with 35-foot penetration in the time given. On the entire round piling the average was 70 piles per day per driver.

Making the Wakefield sheet-piling went on night and day from the time the men and material arrived at Hymelia until the entire 1,116 were completed. Wakefield sheet-piling was made with 3 by 12 inch, in three layers, with a tongue of 4 inches. Piling of

Fig. 3. Crevasse from South End Looking Toward River. Shows how the eddy on land side erodes levee when river side was held with canvas. (See p. 66.)

this class were 40 and 50 feet in length and were made by gangs of eight men—two carpenters and six laborers. Originally, the plans called for this piling to be bolted and joints were to have a 4-foot lap, but these plans were changed so that the bolting was eliminated and in their places 9-inch wire spikes, clinched, were used. Each gang averaged 30 of the 40-foot piling per day.

An electric plant was installed for the purpose of furnishing light for the night work. From the time the work started the day crews were worked from daylight to dark. Most of the labor used on the work was brought up from New Orleans, local Negro labor being used for such work as unloading lumber, etc. Wages paid were at the rate of 15 cents per hour for labor, 30 cents per hour for carpenters and bridge men, and 50 cents for foremen.

In driving sheet-piling, the best record was 122 in a day, while the average was 80 per day per driver. When these drivers approached the closing, the current had increased to 12 miles per hour. All of the drivers, except the track driver, were pulled to one side as the center of the dam was reached and the track driver completed the closing, working continuously from the morning of August 3, with the same crew, until the last sheet-piling was driven at 9 a. m. on August 4. A penetration of 25 to 30 feet was given on the sheet-piling.

Figuring the round piling as 10 by 10 inches, approximately 900,000 feet board measure were used in the dam. All material was brought up from New Orleans in twenty-one barges. The cost of the dam to the levee district was, in round figures, $34,000 for material and labor, $1,200 bonus and $8,300 commission. As the next lowest bidder would have received $49,500, the levee board saved $7,200 by letting the contract on a commission basis. There was no hitch in the work at any time, as each foreman was instructed as to the work he was to do, and all of the different crews hinged together like the work of a big machine. At no time was it necessary to send to New Orleans for anything that had been forgotten. All of the arrangements for prosecuting the work were made under the direction of Mr. William H. Williams of the contracting firm, Doullut & Williams.

For the benefit of those engaged in similar work, the actual specifications for the timber work are given herewith:

The construction shall consist of a timber dam crossing the crevasse at a point indicated on map, for such length as may be required by the State Engineer in charge, approximately 1,050 feet, said construction to conform with plans and drawings.

The Dam shall be built in accordance with the plans and drawings accompanying the specifications prescribed by the Board of State Engineers; requiring square or round piling, braces, waling, sheet-piling, all of merchantable pine, sound and free from all defects impairing strength of structure, with bolts, nuts, washers and spikes, all of which shall be furnished and placed in position under the following specifications.

Piles shall be either square-hewed or round; they shall be 12-inch at butt end and not less than 8-inch at small end. They shall be driven or placed in position in such a manner as to prevent shattering, splitting or brooming; they shall be driven plumb, in true line, and spaced as shown on drawing. The piles shall be of such length as determined by soundings and cut off at an elevation of 35.0 Cairo Datum, and shall have sound head, not split nor broomed, nor cracked. The piles in each row shall be spaced longitudinally 5 feet between centers and the rows shall be spaced transversely 10 feet between centers. All piles shall have a penetration of not less than 25 feet below the surface of the ground.

Braces. Sawed, sound and square edged; they shall measure 8 by 10 inches; they shall be carefully bolted on piles as shown on

plans; they shall be about 14 feet long, length to be determined by the water surface.

Ties. Sawed, sound, square edged; they shall measure 8 by 10 inches, one on each side of pile, carefully bolted on piles as shown on plan, and shall be 11 feet long.

Sheet-piling. Sawed, sound, square edged, planed on inner surface; shall be of such length as determined by soundings; driven to an elevation of 35.0 Cairo Datum, and have a penetration not less than 20 feet below the natural surface of the ground; they shall consist of 3 pieces 3 inches thick, and 12 inches wide, closely and accurately bolted and spiked together as shown in detail drawings, the tongue to project 4 inches on one side with groove of the same depth on opposite side; they shall be carefully driven, plumb,

Fig. 4. Sand Bag Spur Protecting Levee on Land Side from Reverse Current of Eddy. (See p. 66.)

with close joints, top flush with top of piles, and not cracked, split nor broomed; bolted to waling piece (see detail plan, p. 59).

Bolts, Washers and Spikes. Shall be of the best quality of wrought iron, perfect in all respects, with nuts, washers and screws of United States standard dimensions; all screw threads shall be cut of sufficient length to permit tightening up of screws without the use of extra washers; there shall be a cast-iron washer of good quality and pattern at each end of all screw bolts, not less than 3 inches in diameter; bolts for sheet piling shall be standard ⅝-inch carriage or machine bolts with accompanying nuts and washers.

Workmanship shall be of the highest standard, subject to the approval of the State Engineer in charge.

Discussion

Mr. T. G. Dabney*
Member American Society of Civil Engineers

My own experience in dealing with breaks in the levee has been confined almost solely to the upper part of the river; that is, along the Yazoo Basin front. As a result of such experience my conclusion is, That it is unwise to attempt the closure of a crevasse unless it can be done in the incipient stage of the flow through the opening; or, as the surgeons say, the wound can be healed "by first intention."

If the work must be done with deliberation, and time consumed in assembling plant and material, the usual result is, that by the time of completion of the work of closure the flood has receded and the closure accomplishes nothing but a waste of money.

Occasions have arisen in my experience when it was found expedient to cut off the flow through the crevasse on a receding flood, where the physiographical features rendered the work feasible and the cost moderate, thereby saving valuable time for the landowners in the seeding season.

But the earth formations in the upper part of the river's course, and this should probably apply to all above Red River, is favorable generally to a very rapid widening of the opening through the levee, and deep gorging of the land in the track of the flow; conditions very unfavorable to the operations of crevasse closing.

In the territory in lower Louisiana the earth formation is much more homogeneous and of a more tenacious character than obtains in the regions farther up the river; and some successful attempts have been made in that region to close crevasses; but whether the cost was economically justified I am not informed.

The only method of crevasse closure that has been successfully employed, in the more deliberate operation, is first to build a skeleton work composed of small hand-driven piles, four or six to the "bent," so disposed as to minimize the obstruction to the flow of the crevasse water through the structure. Upon this structure is then placed a walkway of plank, to give ready access to all parts of the work at the same time; then a revetment of earth sacks is placed on the ground between the rows of piles, consisting of one thickness of the bags, and this is extended outside of the structure to catch the overpour. The further operation is the placing of additional sacks within the structure so as to bring it up uniformly in height throughout its entire length, allowing no concentration of flow at any point. This process is continued until the top of the sack levee surmounts the surface of the flood water and the flow is entirely cut off. If available, brush may be distributed uniformly along the upstream side of the piles, to moderate the velocity of flow.

*Chief Engineer Yazoo-Mississippi Delta Levee District.

But the essential principle to be adhered to is, by a rapid distribution of materials along the whole length of the structure to bring it up as nearly as practicable with a uniform top level, so as to avoid concentration of flow with resulting scour of the ground behind the dike.

This seems a rather meager treatment of the subject, but embodies about all that is to be said in a general treatment of it.

Capt. C. O. SHERRILL
Corps of Engineers

Hymelia Crevasse (figs. 3, 4, 5, 6, 7, 8, and 9) occurred May, 1912, at 7 p. m., but inasmuch as the United States does not share in the care of the levees after they are built, and has never before

Fig. 5. Canvas Ready for Placing on End of Levee. (See p. 66.)

undertaken the closure of a large crevasse, no material or complete organization were on hand ready for the work of closing the break. Due to this and other related causes, construction work was not started until the crevasse was about 800 feet wide with the levee ends caving at the rate of 200 feet a day. This delay allowed such an extensive erosion of the batture in front of the crevasse that the closure was found to be impracticable until the river should have fallen to a more normal stage.

The method followed in the attempt to close the crevasse was to build, at a point about 780 feet from each caving end, open cribwork across the batture so as to pass jaws of crevasse at a distance of about 400 ft. (Figs. 6 and 7.) The cribwork consisted of 5 rows of 4 by 6 inch timber piles, spaced 8-foot centers transversely and 4 or 5 foot centers longitudinally, driven to a penetration of at least 12 feet. For driving piles, improvised pile drivers were used

for three days while two batteries of four pile drivers each, with 250-pound hammers, were being made (photographs show the crevasse and details of crib and pile drivers).

Rapid progress was made with these latter, about 250 feet a day being accomplished. The cribwork consisted of 2″ by 12″ roadway bearers drifted to piles, and further supported by 2″ by 6″ longitudinal stringers to stiffen the structure and to help carry the load of sacks later to be placed on the crib. A 3-inch flooring was securely nailed to the stringers to cover the entire upper surface, except where openings were necessary for dumping sacks. The crib was further strengthened by triangular braces on the two downstream rows of piles, placed in such a manner as to act as a truss and independent of scour of the batture. It was planned to haul out filled sacks on the cribwork with dummy cars and mules, and load them on the decking in sufficient quantities to floor the entire crib simultaneously as soon as interlacing should be completed. About 1,000,000 filled sacks were required. The interlacing consisted of 2 by 4 inch timbers spaced 12 inches center to center, driven to a penetration of at least 8 feet. These interlacing piles were driven along the front of each open crib to hold the sacks against the force of the current.

The method used in holding the ends of the crevasse against erosion consisted in placing heaviest doubled canvas sheets around the ends of the levees. These sheets were weighted down to the base of the levee with railroad iron placed longitudinally, the canvas then being made fast by chains sewed to its upper side and inner end. This method had been tried with success at the Torras Crevasse, where the soil was largely a clay, known as "buckshot;" but here, where the soil was light silt and sand, the plan was successful only to the extent of reducing the caving from 200 feet a day to 10 or 20 feet a day, until the barrow pits were washed out to a depth of 30 to 40 feet. The work advanced rapidly except for the delay due to securing pile drivers and materials, but four days were practically lost before these and sufficient plant were ready for work.

On May 23, due to erosion along the barrow pits, 240 feet of the north end of the levee gave way in about two hours time; and on May 25, 200 feet additional went off practically at once, taking the end of the crib with it. These causes, together with the rapid caving of the batture toward the river along the lower channel, led to the abandonment of the work on May 25. There was expended all told on this work, by the United States, by private interests, railroad companies, lumber companies, and levee board, approximately $38,000.00.

TIMBER CRIB DAM.

About two and a half months after the occurrence of the crevasse, the State Board of Engineers of Louisiana let the contract for the timber crib dam described in the article "Methods and Cost of

Fig. 6. Showing Construction of Battery of Four Pile Drivers and the Character of Open Crib. (See p. 65.)

Damming the Hymelia Crevasse in the Mississippi River." This timber crib dam was built to allay the fears of the residents of the affected district that the water would continue flowing through the crevasse until late in the fall. The dam had no effect on the work of replacing the levee, except to decrease slightly the spread of dredged material on the river side of the fill (see cross section, fig. 9, p. 71).

The author of the article in question inadvertently makes several statements which give a wrong impression of actual conditions. He says: "The flow of water had at that time dug for itself a channel 1,100 feet wide and 18 feet deep, the river being at 32.2 feet Cairo Datum. Mean Gulf Level corresponds to a reading of 21.56

Fig. 7. Temporary Protection Crib for Holding Levee End. (See p. 65.)

Cairo Datum, which would mean a head against the dam of 10.64 feet, in addition to the current, which at that time was approximately 5 miles per hour." It is also stated that "When these drivers approached the closing, the current had increased to 12 miles per hour." The survey party working at Hymelia measured the head against the dam immediately after it was finished and found this head to be *15 inches* instead of 10.54 feet. A party of engineers inspected the work when it was two-thirds finished and found the water apparently without current and at the same height on both sides of the dam. In fact, the dam, though an excellent piece of construction in still water, and well executed, was in no sense a closure of a crevasse.

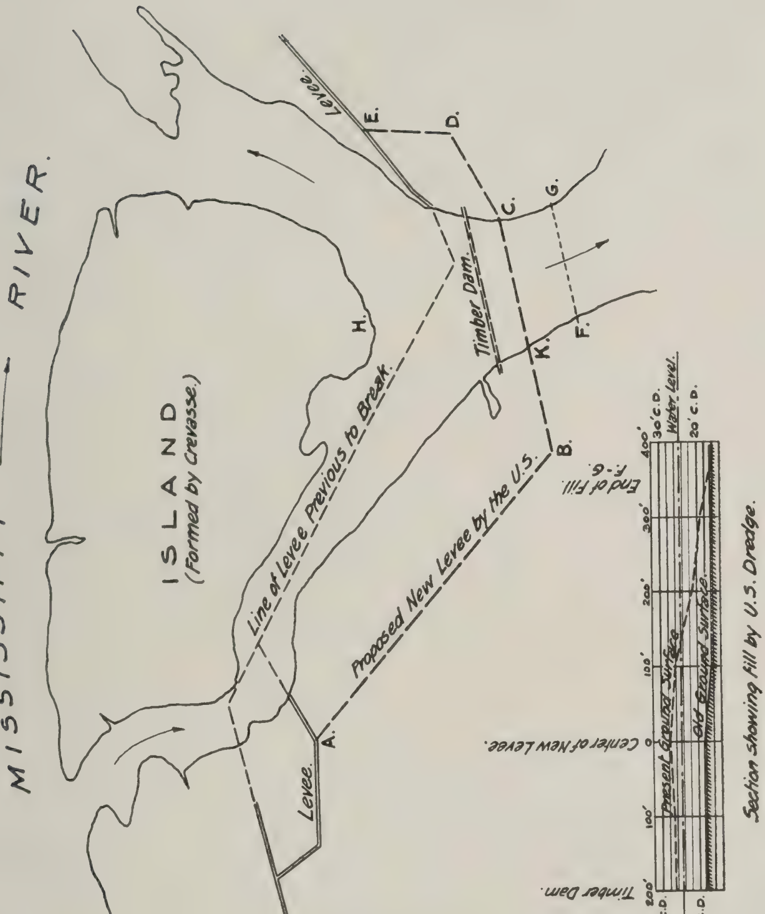

MISSISSIPPI —— RIVER.
ISLAND
(Formed by Crevasse.)
Line of Levee Previous to Break.
Proposed New Levee by the U.S.
Levee.
Levee.
Timber Dam.
A.
B.
C.
D.
E.
F.
G.
H.
K.
End of Fill F-6.
Center of New Levee.
Timber Dam.
Water Level.
Present Ground Surface
Old Ground Surface
30' C.D.
20' C.D.
200'
100'
0
100'
200'
300'
400'
Section showing Fill by U.S. Dredge.

Fig. 8. (See p. 70.)

REBUILDING THE LEVEE.

On August 20, 1912, the Fourth Mississippi River Engineer District let a contract with Landeche & Lambert Bros. for all of the new levee on line A, B, C, D, E (fig. 8, p. 69), except portion CK, which includes that through the gorge. This contract involves 90,000 cubic yards at 24.9 cents per yard.

On August 1, the dredge *Pascagoula* was secured from Maj. C. A. F. Flagler, Corps of Engineers, for the work of filling in the gorge to form the base of that part of the levee through the channel. The dredge worked for twenty-one days and placed 76,000 cubic yards net in the gorge, pumping the earth from the point H on the island formed by crevasse. The section (fig. 9, p. 71) shows the result of the dredging along a typical section of the levee. On account of a large quantity of silt which had been deposited in the gorge, it is estimated that the dredge actually moved over 100,000 yards of earth to give a net of 76,000 yards after settlement. The total cost of this work, including all expenses of towage of plant from Gulfport to Hymelia and return, a distance of 468 miles, was $12,000.00, or less than 16 cents per yard.

The *Pascagoula* could not be spared longer for the work from the Mobile office, so it became necessary to let a contract for the remaining 80,000 cubic yards of levee on the base thus prepared. The Southern Dredging Company was given the contract, on September 18, at 54 cents a yard, the only other bid having been 78 cents. Informal estimates had been received from contractors on building the levee across the gorge before any dredging was done, and none were less than $1.00 per yard, with no offers to take it at that rate. By using the dredge for filling the gorge, instead of letting the work as a whole by contract, the Government saved about $90,000.00, or about $52,000.00 more than the total cost of the attempt to close the crevasse.

GENERAL REMARKS ON THE CLOSURE OF CREVASSES.

Each time disastrous floods occur on the Mississippi River, the question comes up as to the advisability of attempting to close crevasses. The reasons generally given for not attempting their closure are: 1st. The great difficulty and expense of the undertaking, even if successful; the probability of failure, and the great damage that will have been done before the closure is completed.

The chief reason for failure in closing crevasses in the past has been that no previous plans have been made for the work, no materials and plant have been collected, and no organization provided ready for the emergency. The result has been that several days delay in getting started occurs, the width of opening becomes greatly enlarged, and the erosion of the gorge immensely increases the difficulty of the work.

Up to the present, too, no successful effort has been made to improve the system of closure heretofore used—an open-crib structure

floored throughout by the simultaneous deposit of sacks of earth. To close a crevasse by the present methods the entire crib must be finished before the sacking begins; otherwise, erosion of the soil is so rapid, due to a partial obstruction of the current by the sacks placed at one spot, as to cause the piles to be quickly undermined and the structure to be destroyed. When the entire crib has been floored with sacks of earth, it is possible to elevate the sack filling by successive layers without causing extensive erosion. It is evident that in water 10 to 20 feet deep an enormous number of sacks must be used to fill the four or five rows of cribs necessary to the stability of the structure under a head of 8 or 10 feet, and against a 12 to 15 mile current.

The only other method of closing crevasses, seriously suggested up to this time, is the use of timber sheet-piling driven from each end, as in case of the timber crib. This has never been used successfully against heads of more than 5 or 6 feet, on account of the

ELEVATION.

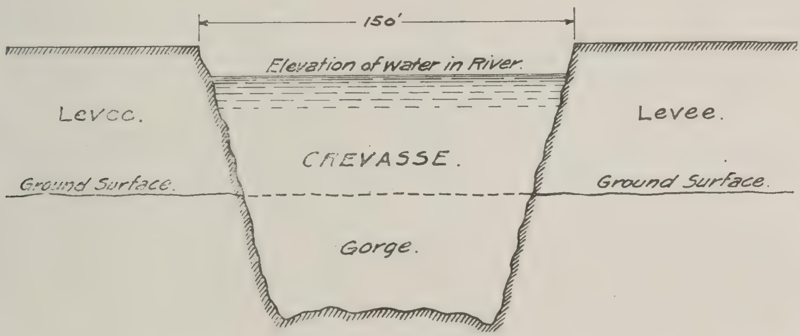


Fig. 9. (See p. 70.)

swift current and enormous erosion caused as the flow is reduced, due to the obstruction of the sheet-piling. This difficulty will be appreciated by any one who has ever seen the rapids below Niagara, as a large crevasse is quite as turbulent (see photographs).

Fig. 10 (p. 73) shows type of closure that appears to me to be possible at small cost and with reasonable certainty of success. The first essential in the work is to have two complete independent units fully equipped on barges, with supplies of steel sheet piling, steam pile drivers, and all supplies necessary for the work. These barges would be moved to the two ends of the crevasse and the construction of the sheet-piling caissons would be begun (as indicated on fig. 10 for holding the ends of the levee. It would be necessary to extend the land side of these caissons until there would be no danger of the erosion cutting the levees in rear of the caissons. If the sheet piles were driven to a penetration of about 20 feet at first, it would be sufficient. Later, as the earth at the ends of the levee eroded enough to threaten the caisson, the resist-

ance to driving would be so decreased that these piles could be driven as deeply as required.

As soon as caissons were finished, pile drivers would start driving the line of sheet piling marked A, B, (fig. 10, p. 73), the pile driver being supported on round piles driven as it advanced across the crevasse. This first row of sheet piles would be driven practically flush with the original ground surface in order not to obstruct the current and thus increase the scour. As soon as this row was completed, working from both ends, a second row would be driven to a height of about 3 or 4 feet above the first; because, having the first row placed, the fall of water between the two rows would not cause undue scour and also the small head would be easily supported by the sheet piles. Similarly, a third and, if need be, a fourth row would be driven, until the last row extended above the water. The location of the lines of sheet piles would be varied according to current and soil; in good clay they could run straight across. A hydraulic dredge would then be put to work filling behind and over the sheet-piling to rebuild the levee. The sheet piles could be removed for future use as soon as the levee in rear was finished.

The greatest difficulty in this work would be to drive the round piles for the pile driver platform in the swift current, but as there would be no obstruction to flow other than the levee ends, this should be feasible. The work of driving the sheet piling would involve only two sheet piles being opposed to the current at a time, for as soon as the one in advance was driven to a sufficient penetration, the next one in rear would be then driven flush with the original ground surface—using a follower. The above system can, in my opinion, be used successfully in all cases where the soil is of clay and probably in all kinds of soil, but with greater risk where silt and sand are found.

ADVISABILITY OF ATTEMPTING THE CLOSURE FROM AN ECONOMIC POINT OF VIEW.

The majority of engineers seem to be of opinion that even if a crevasse could be closed it would not pay, because most of the damage would be done before the work could be completed. With this view I do not agree, because, assuming that the closure would require three weeks at the outside, this would mean that the water would be on none of the land much longer than that time and the extent of territory covered would be small as compared with that covered if the crevasse were allowed to flow for three or four months as usually happens in such cases. I believe the closure could generally be effected in less than two weeks.

The losses from crevasses in the Mississippi Valley are largely due to the destruction of crops and live stock, but most of all to the fact that planters are prevented from replanting by the length of time the water remains on their fields. To see the frightful rush of waters over the country, one naturally thinks everything

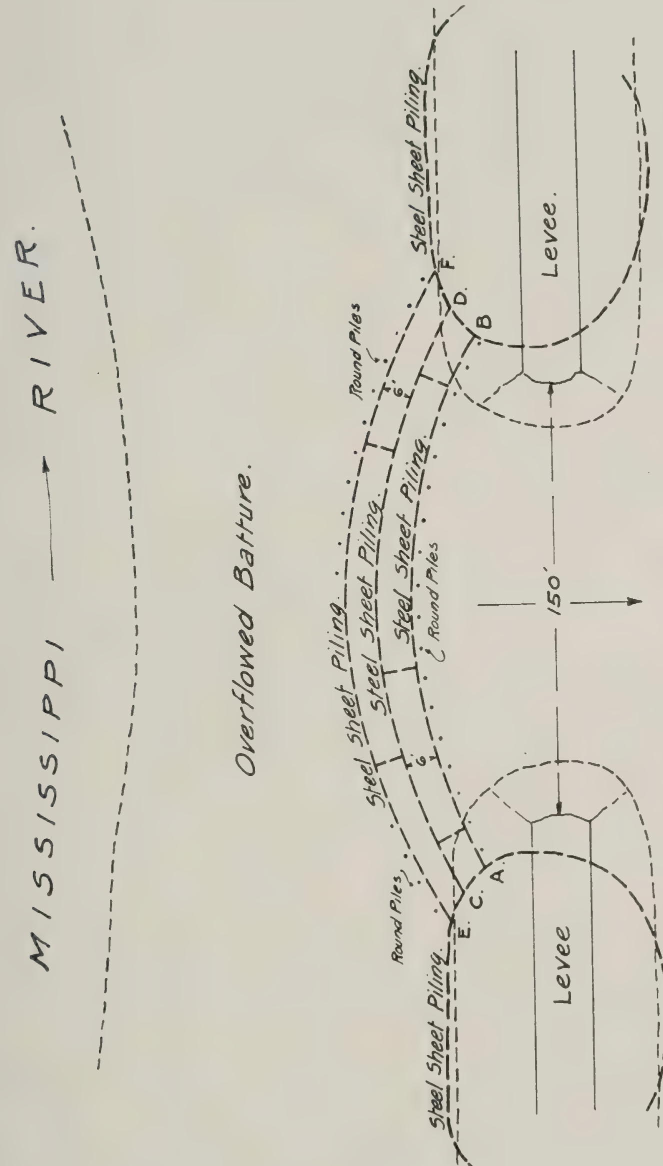


Fig. 10. (See p. 71.)

is lost, but the actual damage to buildings and improvements, except above noted, is relatively small. Residents could even remain in their houses and care for their stock for two or three weeks, if assured that the flow would be stopped. A large percentage of the lands could be protected from overflow by the present systems of back levees, if the elevation of the crevasse water were limited by a two weeks flow. A crevasse closed within two weeks would usually reduce the losses to one-fifth or one-tenth of the total that would occur if it were left unclosed. The resultant saving would amount to millions of dollars for each crevasse closed, so why not spend some money on previous preparation for an attempt which promises so much if successful? The Hymelia Crevasse was flowing until about August 1, or two and a half months after it occurred, and replanting in the affected territory was impossible. The losses from crevasses in Louisiana this year are estimated at $25,000,000, of which about half was due to breaks in Arkansas.

In conclusion, this is an engineering problem which promises probably larger returns on the investment for successful solution than any other ever presented, and I, for one, am convinced that it can be solved by careful preparation beforehand, combined with organizations practiced for a few weeks in their duties.

Mr. Luther Y. Kerr

Assistant Engineer; Member American Society of Civil Engineers

This article appears to contain several important inaccuracies, the correction of which is necessary, to the end that the reading public may have the benefit of more exact information than it affords in its present state; which, however, for casual reading, offers an interesting insight as to the extent of levee crevasses, as well as of the difficulties and cost of their repair or control.

The author, speaking of the break, states that:

"The Government attempted to repair it shortly afterwards, but a second flood wiped out about $60,000 worth of work and compelled the Government engineers to abandon the task."

From an examination of the gage readings at College Point, about 25 miles above the break, we find that there was not the slightest upward oscillation of the flood between May 14, the date of the break, and August 4, the date of the completion of the closure.

Captain Sherrill states in his "Report of operation" for the month of May, concerning the Government engineers' attempt to close the gap:

"The actual work of driving piles for cribbing and protecting the ends of the levee to prevent further caving was begun May 18, and was finally abandoned May 25, due to failure to prevent erosion of the ends of the levee and the rapid deepening of the channel through the crevasse."

The difficulties reported by Captain Sherrill are precisely the troubles that may, uniformly, be expected in such undertakings.

When a flood, with a head of 10 or more feet, breaks through the levee, the current created by such head is obviously very great, with the result that its impingement upon the natural adjacent soil, frequently of a light loamy character, causes immediate and extensive erosion or scour, usually covering the entire width of the break, and extending for several hundred feet inland.

These crevasse erosions, or "Blue Holes" as they are commonly called, frequently exceed 50 feet in depth below the natural surface. Under such conditions one may readily understand that any local obstructions placed in or near the crevasse, under such current conditions, especially if it disturbed the then existing soil equilibrium, as would naturally result from the driving of piles, would greatly aggravate the situation and increase the rate of scour.

The author states that the levee at the break was 15 feet in height, which checks closely with the levee profiles of record in the United States Engineer Office at New Orleans.

By comparing the flood heights of 1897 with that which broke the Hymelia levee, and applying the difference at that point, we find that there could probably not have been more than 13 feet of water against the levee upon the date of the break, May 14, 1912. On that date the College Point gage read 30.1 feet, and the Carrolton gage read 20.6 feet. On July 20, when the closure by the State authorities was initiated, the College Point gage read 12.6 feet and the Carrolton gage read 8.3 feet, giving a fall of 17.5 feet at College Point and 12.3 feet at Carrolton.

Allowing for the flattening of the slope and correcting to the location of the crevasse, a fall of 15.3 feet between the above dates. is indicated for that locality, from which it appears that the flood plain on July 20, at the Hymelia Crevasse, was 2.3 feet below the natural banks of the river, at which stage this second effort at a closure was begun. By the time this closure had been completed, August 4, there had been a further fall of 4 feet, making a total fall of 19.3 feet, and bringing the flood plain down to 6.3 feet below the natural surface.

These deductions are, in a measure, borne out by the exposed bank in evidence in the background of the illustration published with the article under discussion.

Now, with the river flowing 6.5 feet below its natural banks, was not the final work on this structure more in the nature of wooden-dam-construction across a lake?

It is true that the adjacent country slopes away from the river, but a careful inspection of the topographical maps of this locality will indicate that it is apparently about three-fourths of a mile from the crevasse back to a contour over which the river at that stage could flow.

The manner and rapidity in and with which labor, machinery, and materials for this work were assembled, the organization that

was perfected in so short a time under the most trying circumstances, and the systematic manner in which actual construction progressed are all a matter of great credit to the ingenuity and resourcefulness of the man or men "Behind the Guns." *But, did it pay?*

What was the actual damage being done by the discharge through this crevasse August 4, with the river flowing 6 to 7 feet below its top banks?

The class of construction customarily employed in attempting crevasse closures can rarely, if ever, be subsequently incorporated in the permanent repair, and certainly the large expenditures involved are not warranted simply to appease a public clamor for "something" to be done when the engineers are satisfied that relief will come in a natural way before, or by the time, it could be accomplished otherwise.

In 1890, the Mississippi River Commission passed the following resolution:

"That it is the judgment of the Commission that it is, as a rule, an unwise expenditure of money to attempt the closure of flowing crevasses."

The writer has seen in action a great many of the larger crevasses, and the result of practically all of the crevasses that have occurred between Cairo and Vicksburg, during the past twenty years, and he has observed no conditions that, in his opinion, seemed to discredit the wisdom of that resolution.

Now, inasmuch as an elementary knowledge of the best methods of preventing crevasses should be conducive of much better results than the most advanced theories of how to close them, I would like to touch lightly, in passing, on the method employed to stop the "mud boils" that developed behind the levee.

It is stated that "an attempt was made to stop the damage being done by loading the ground over the supposed hole with sand bags." "The object of this loading of the levee was to crush in the earth around the hole." The application of such methods under such conditions is, in the writer's opinion, little short of courting disaster

When seep water appears on the back slope of the levee to such an extent that sloughing is threatened, the problem at hand is not to prevent the water from passing out, but rather to permit of its passage and disposal as readily as may be and, at the same time, hold the levee material in place.

This is usually accomplished by first laying a mattress of brush, or other material, such as to afford free drainage, over the threatened slough, and then heavily weight such mattresses with sacks of earth.

In the case of sand, or mud, boils back of the levee, perhaps the safest and most practical method is to build a wall around the threatened area with sacks of earth, allowing the well or inclosure so formed to fill up with water discharged from the seeps and

boils, thus creating counter head, with diminished seep water velocity, and affording an opportunity for the seeps and boils to choke themselves out by way of deposited, or slowly infiltrated, sand. The water impounded in such case need not be very deep, 2 or 3 feet would be ample, and it can then be allowed to flow over the wall or out through a spillway.

Caisson Work at Hales Bar Dam

BY

E. L. MADERE AND ROGER B. McWHORTER
Junior Engineers

Hales Bar Dam is located in the mountain section of the Tennessee River, 33 miles below Chattanooga, Tenn. The contract for this project was let in September, 1905, but work did not begin on the dam proper till November, 1909. This dam is being constructed for the dual purpose of power development and river improvement. The river between Chattanooga and Hales Bar contains a series of swirls and rapids, and at low stages is impassable to steamers; this condition will, of course, be remedied when the dam is completed. The dam is 1,200 feet long, joining the powerhouse on the left bank and the lock on the right bank. Its height varies with the elevation of the foundation, as does its width, the back being battered three horizontal to four vertical, while the upstream face is vertical. The maximum height thus far is 76 feet. The crest of the dam is at elevation 636, low water at Hales Bar being 598.5 feet above mean Gulf level. Its cubical contents are estimated to be 100,000 yards of concrete, of which 60,000 yards have been placed.

The foundation rock is a badly seamed limestone, having been cut into numerous boulders of varying sizes by the action of water. This leaky and creviced rock made deep excavations necessary to secure foundation, and the original idea of securing all the foundation in the open had to be abandoned, after 544 linear feet had been secured. The theory was advanced that the water, which gave so much trouble in the coffers, came from mountain springs, but an experiment proved conclusively that the water came from the river. Potassium permanganate was put in bags and sunk to the bed of the river above the upper cofferdam, and a trace of it was easily detected in a sample taken from the pit at elevation 562. As the work progressed toward the middle of the river, from both sides, the water, due to the leaky and creviced rock, grew more

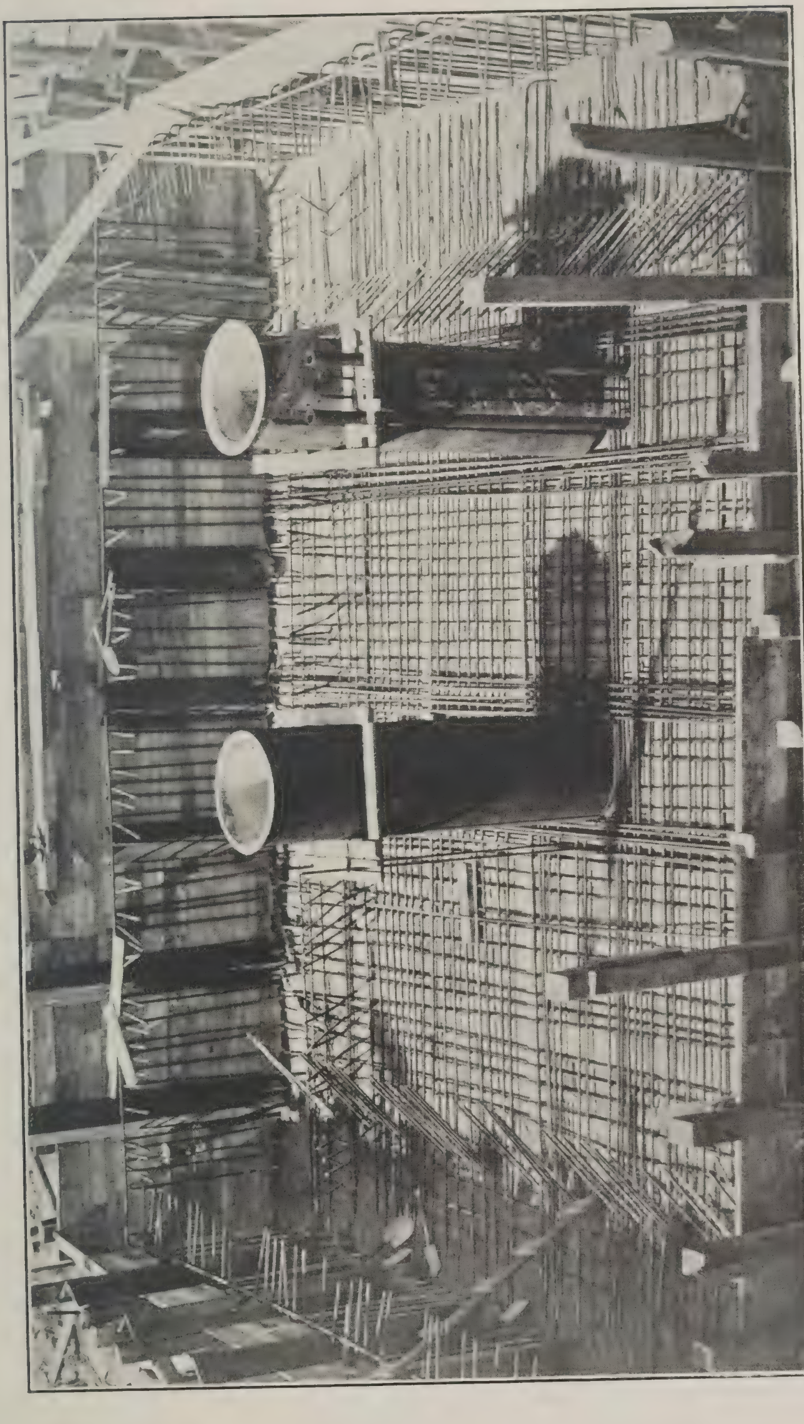


Fig. 1. Steel Reinforcement of Concrete at Bottom of 40 by 40 foot and 30 by 32 foot Caissons. This also forms the roof of the working chamber. Similar reinforcement is placed 6 feet above that shown in the illustration.

difficult to control, and it finally became apparent that the open-air method was no longer feasible. Consequently, the use of pneumatic caissons was permitted by the United States Government, July 27, 1911, and their construction began August 11, 1911. The accompanying illustration (Plate I) shows relative amounts of foundation secured by the two methods and amount yet to be secured by the caisson method.

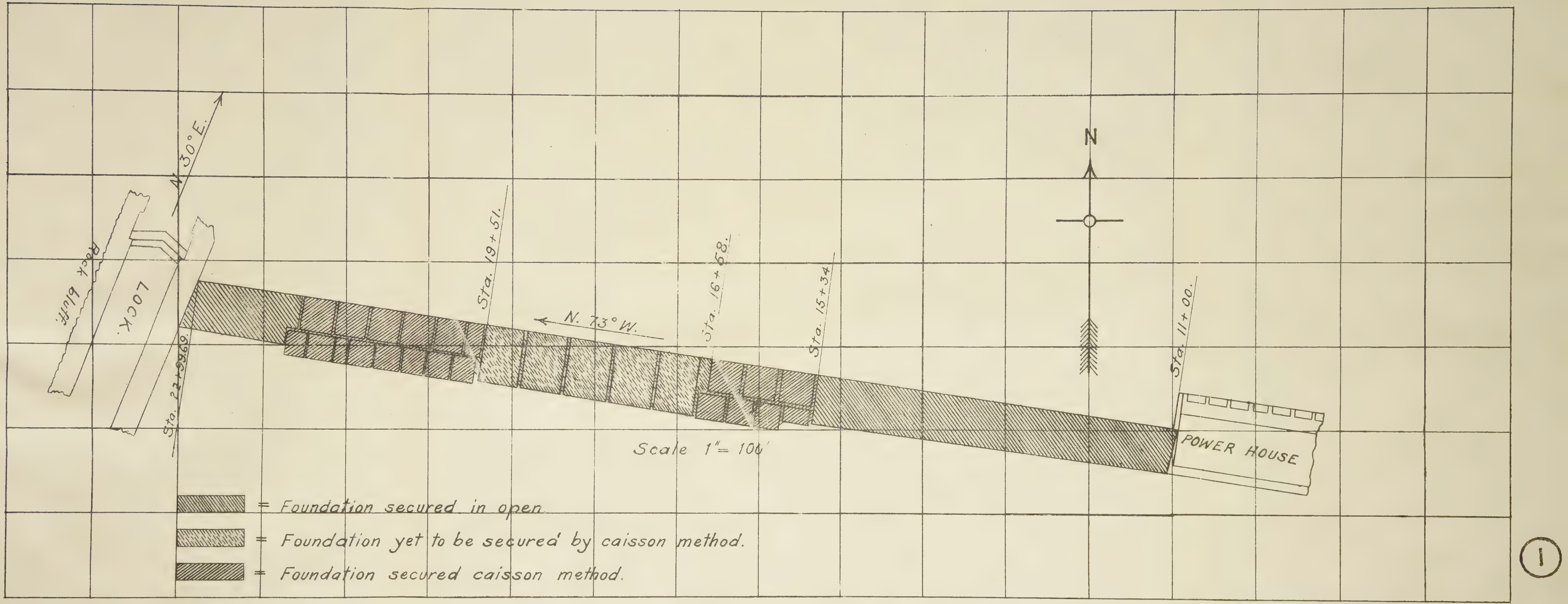
Two rows of caissons form the base of the dam where the caisson method has thus far been employed. The size adopted for the face caissons was 40 by 40 feet; and for the toe caissons, 30 by 32 feet. The first set of caissons were placed 1 foot apart, but much trouble was experienced in sealing the spaces between them, and those built later were placed 18 inches apart.

For the remaining caissons to be placed, it has been decided to increase the breadth across the current to 54 feet and the length in the direction of the current to 70 feet, and eliminate the toe caissons. There will be five caissons of this size and they will be placed 3 feet apart. All caissons are built of concrete with a steel cutting edge, and are sufficiently reinforced with 1-inch twisted steel rods to prevent any likelihood of a collapse. The illustration (fig. 1, p. 79) will give an idea of how this reinforcement is placed. Each of the 40 by 40 and 30 by 32 foot caissons have two circular steel shafts, 3 feet in diameter, extending through the roof, one forming the manlock and the other the mucklock. The 54 by 70 foot caissons will have four mucklocks and two manlocks.

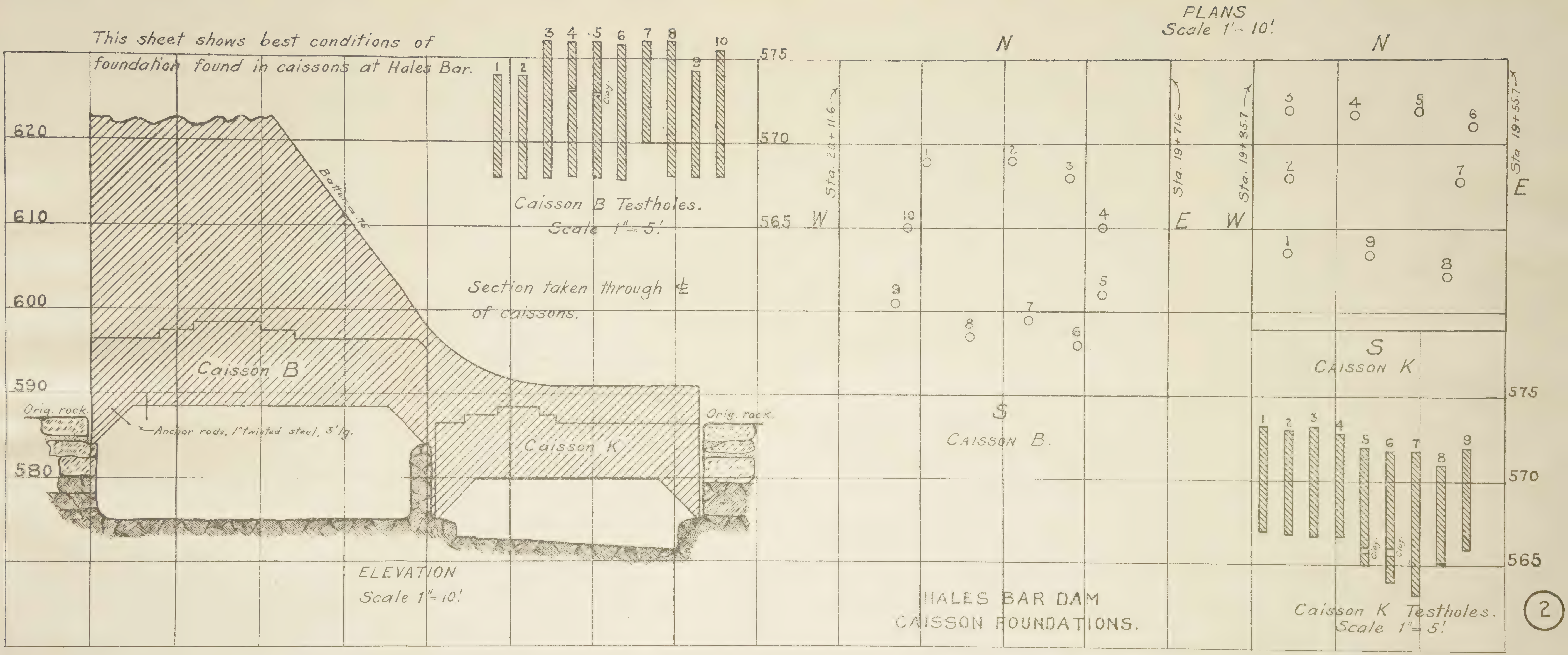
The foundation under some of the caissons, after the excavation had been carried to a reasonable depth, evidenced by test borings, showed up as well as any foundation secured in the open; others having gone to a much greater depth, showed very unsatisfactory conditions. (See accompanying illustrations, Plates II and III.) Under caissons, as in the open, the foundation is thoroughly tested to a depth of from 8 to 12 feet by percussion drill holes before concreting is begun and all holes developing crevices or leaks are piped for subsequent grouting, in view of solidifying the foundation and stopping the leaks. The grouting is done from the outside after the interior of the caisson has been concreted, the pipes having been extended through one of the shafts to a convenient height above the caisson.

When the caisson rests in its final position all gravel, sand, and clay is removed from the surface of the rock and surface seams are cleaned as deep as is thought practicable; generally, from 2 to

Sta. 15.



HALES BAR DAM
PLAN OF FOUNDATION



This sheet shows best conditions of foundation found in caissons at Hales Bar.
PLANS
Scale 1" = 10'
N
N
620
610
600
590
580
575
570
565
Batter = .75
Caisson B
Caisson K
Orig. rock.
Orig. rock.
Anchor rods, 1" twisted steel, 3' lg.
Clay.
Caisson B Testholes.
Scale 1" = 5'.
Section taken through ℄ of caissons.
ELEVATION
Scale 1" = 10'.
Sta. 20 + 11.6
Sta. 19 + 71.6
Sta. 19 + 85.7
Sta 19 + 55.7
W
E
W
E
S
CAISSON B.
S
CAISSON K
Caisson K Testholes.
Scale 1" = 5'.
HALES BAR DAM
CAISSON FOUNDATIONS.
2

5 feet, according to the breadth of the seam. The clay and clay-bag seal around the cutting edge is replaced by a wall of cement in bags laid in neat cement mortar. This wall, when practicable, joins the adjacent caissons, and during its construction 2-inch pipes are inserted into the space between the caissons, later to be extended up through the mucklock and used in blowing the material out of the space and in grouting same. All leaky test holes and springs are piped for grouting. The surface of the rock, and walls and roof of the caisson are then washed and the caisson is ready for concrete, which is locked in at the rate of about 20 yards per hour. It is impossible to completely fill the interior of the caisson from below, and the small space that remains near the roof is sealed by grouting through 3-inch pipes, placed through the roof of the caisson during construction, especially for this purpose. For fear an effective bond would not be gotten here and regardless of the fact that the dam is a gravity section, two rows of 1-inch anchor rods, 3 feet long, were placed on the upstream side of the caisson, the rods being 2 feet apart in the row and staggered. These rods are placed during the construction of the caisson, and half their length projects into the working chamber for the purpose of anchoring the caisson to the concrete core below.

The spaces between the caissons are treated by three different methods, those very narrow being treated altogether by grouting; others, where there is room enough for a man to work and where one of the cutting edges is at a relatively high elevation, are cleaned out and filled in the open; still others are covered with a concrete roof and worked under compressed air. As previously stated, the grouting method is particularly adapted to narrow spaces and to spaces where the excavation goes to an exceptionally great depth on both sides. This method is very laborious and tedious. A high-pressure water hose is turned into the space, which gets the clay in suspension, and a syphon running simultaneously throws it out. This is continued until the clay is practically all out. A cap of concrete about 4 feet thick is then built over the top of the space, through which 3-inch pipes are placed, 3-foot centers, for the purpose of subsequent grouting and drilling. When this cap of concrete has set, all foreign material remaining in the space is washed or blown out through the pipes, under high pressure. When the space is satisfactorily cleaned, grout is forced into it under pressure. After this grout has set for forty-eight

hours or more, percussion holes are drilled through the pipes to a depth not lower than the higher cutting edge of the two caissons. This process of grouting and drilling is repeated until any space thus treated takes less than fifteen bags of cement, when it is considered sealed and is accepted.

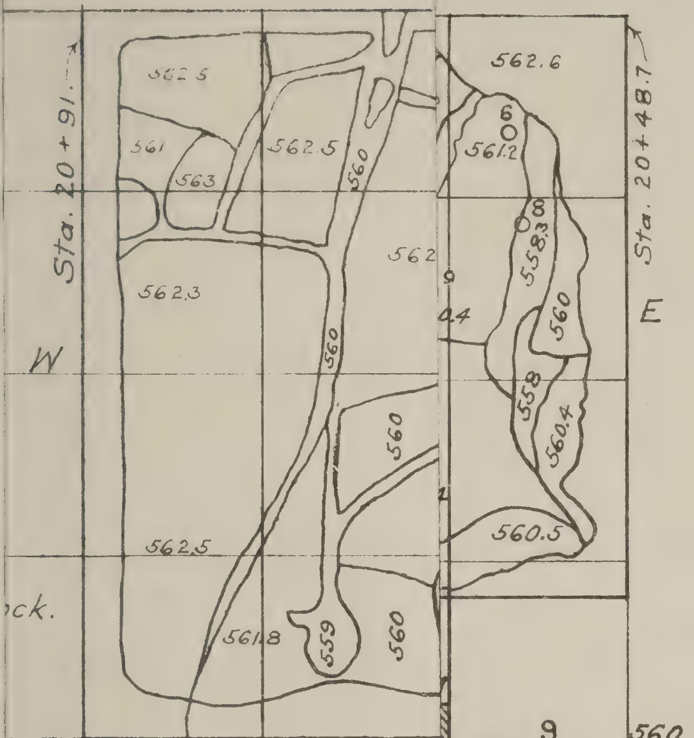
Spaces concreted in the open are thoroughly cleaned down to the rock upon which the higher cutting edge rests.

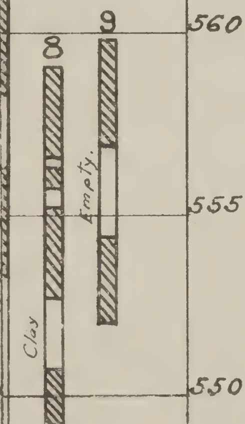
The loose material between the lower cutting edge and the rock can not be removed by hand, and consequently 2-inch pipes are driven to the elevation of the lower cutting edge and extended to a convenient height above the caissons. The space is then filled with concrete, and after it has set a sufficient length of time the remaining material is washed and blown out through the pipes (including those placed under the cutting edges from the interior of the caisson), and is replaced by grout. (See accompanying illustration, Plate IV.)

Sealing spaces under compressed air is very similar to the open-air method but is more expedient, inasmuch as several spaces can be worked at one time. A concrete roof is built over the spaces, with the necessary shafts included, after which the excavation is carried on as in an ordinary caisson.

When practicable, the concrete laid immediately above the caissons is made to break the joints between the caissons 3 feet or more.

In conclusion, it might be stated that the rate of progress of the work since the innovation of caissons has been slightly greater than it was under the open-air method, and this, too, when caissons were used on the worst section of the dam site, therefore necessitating much deeper excavations. The very fact that the excavations under some of the caissons went 25 feet deeper than the excavations in the open, and that 27 pounds of compressed air were required to control the water, makes one realize the infeasibility of other methods; and while it is the opinion of the writers that pneumatic caissons are not ideally adapted to dam foundations, it is thought that the results obtained by the caisson method, as used on this work, will prove satisfactory.





CAISSON

This foundation wa
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M

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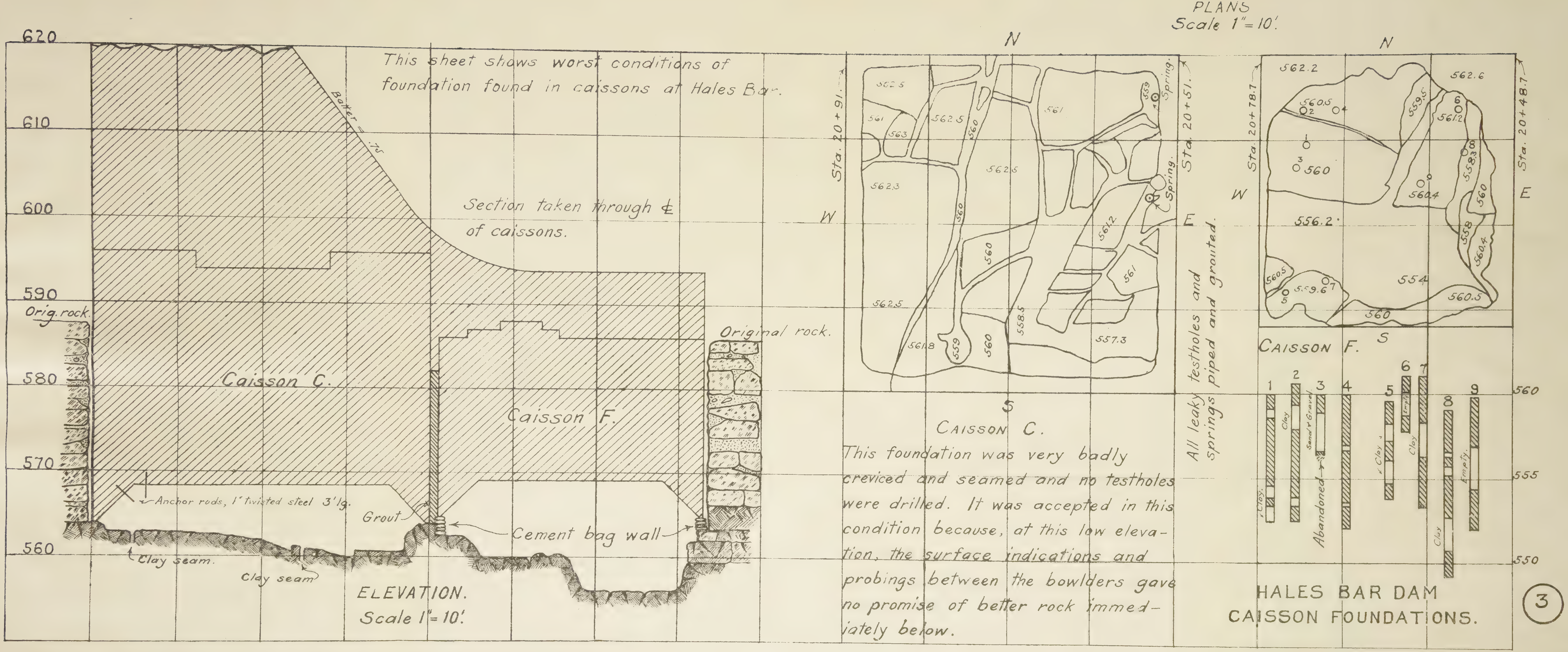
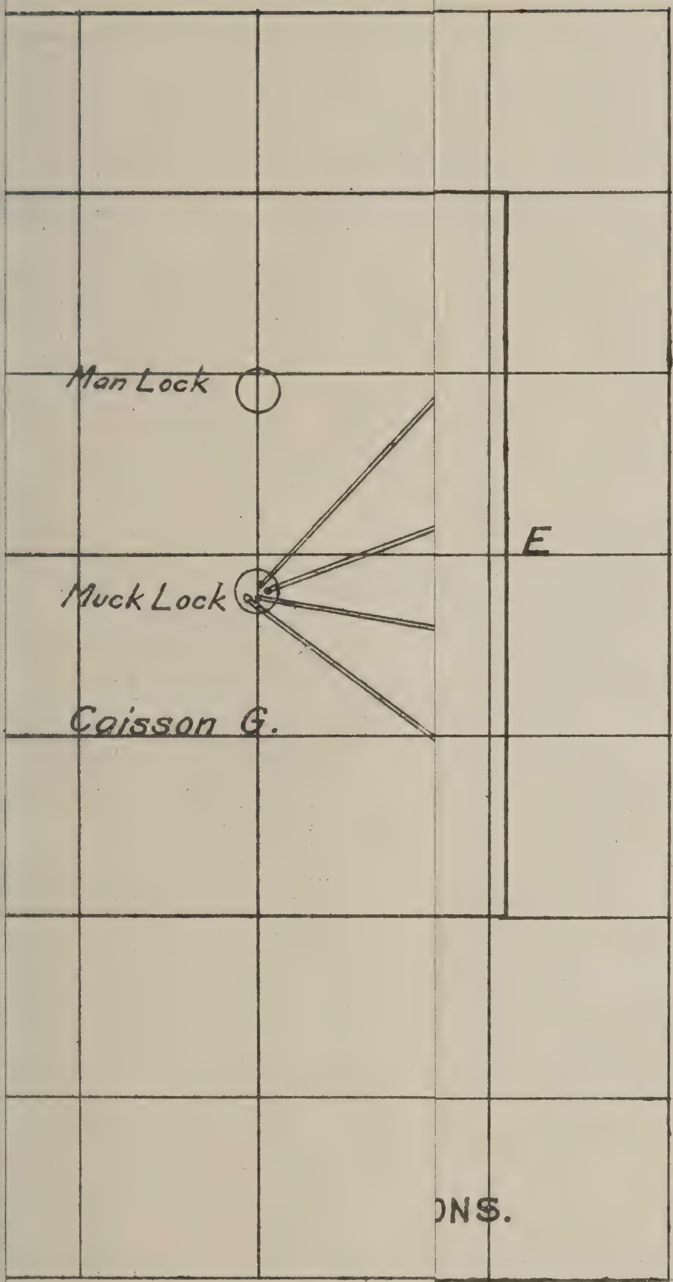
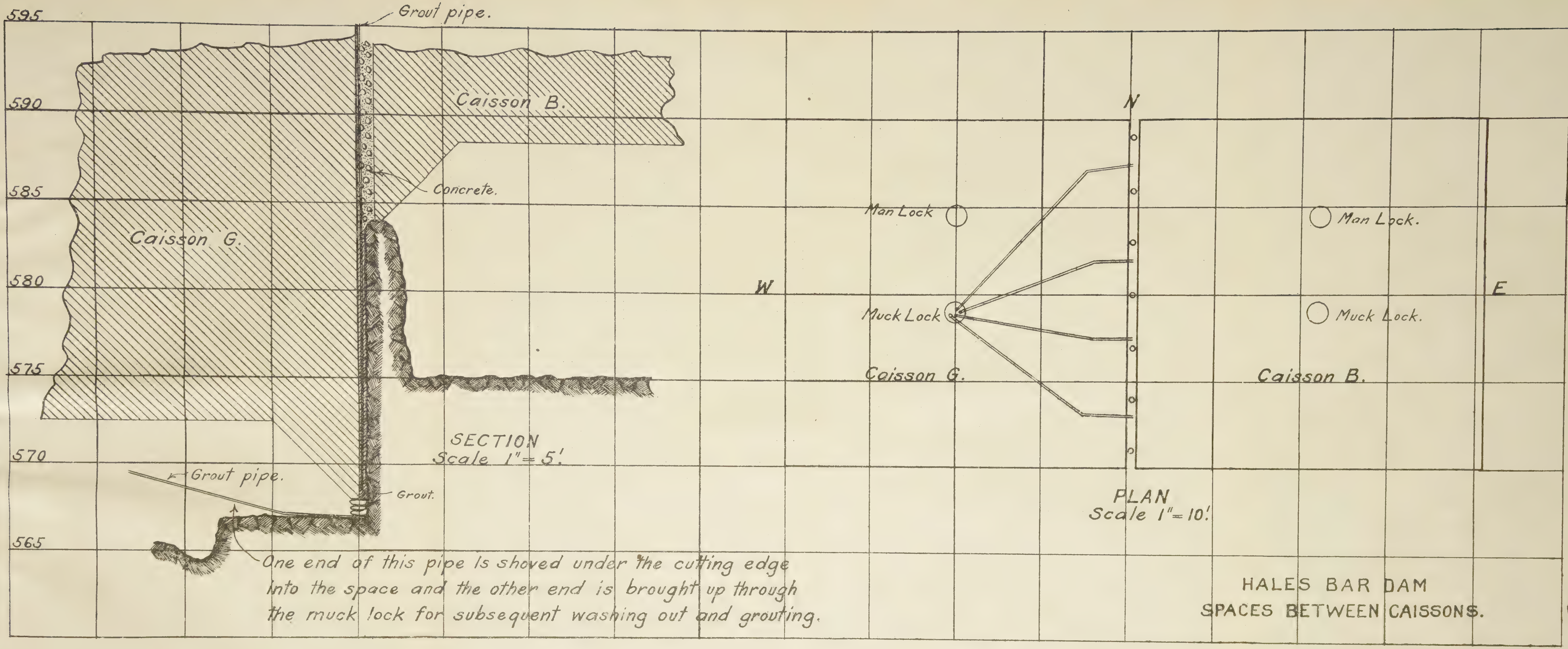
3


PLANS
Scale 1" = 10'.
This sheet shows worst conditions of foundation found in caissons at Hales Bar.
Batter = .75
Section taken through ℄ of caissons.
620
610
600
590
580
570
560
Orig. rock.
Caisson C.
Caisson F.
Original rock.
Anchor rods, 1" twisted steel 3' lg.
Grout
Cement bag wall
Clay seam.
Clay seam
ELEVATION.
Scale 1" = 10'.
N
S
W
E
Sta. 20 + 91.
Sta. 20 + 51.
Spring.
Spring.
562.5
561
563
562.5
560
561
562.5
562.3
560
560
561.2
561
562.5
561.8
559
560
558.5
557.3
559
CAISSON C.
This foundation was very badly creviced and seamed and no testholes were drilled. It was accepted in this condition because, at this low elevation, the surface indications and probings between the bowlders gave no promise of better rock immediately below.
All leaky testholes and springs piped and grouted.
Sta. 20 + 78.7
Sta. 20 + 48.7
562.2
562.6
560.5
559.5
561.2
560
558.3
560.4
556.2
558
560
560.4
560.5
559.6
554
560.5
560
CAISSON F.
1
2
3
4
5
6
7
8
9
Clay
Sand & Gravel
Abandoned
Clay
Empty
Clay
Empty
560
555
550
HALES BAR DAM
CAISSON FOUNDATIONS.
3



4



HALES BAR DAM
SPACES BETWEEN CAISSONS.

4

Biographical Memoir of John Gross Barnard*

BY

Brig. Gen. HENRY L. ABBOT
Corps of Engineers (Retired).

The act of March 3, 1863, incorporating the National Academy of Sciences, contains the name of Gen. John G. Barnard as one of the original fifty members, and his interest in its object and development ended only with his life.

He was born in Sheffield, Mass., on May 19, 1815, being the second son of Robert Foster and Augusta Porter Barnard. His father, the son of Dr. Sylvanus and Sarah Gross Barnard, was a lawyer of marked ability, known beyond the circle in which he lived, although always leading the quiet life of a small town. General Barnard's mother traced her descent from a somewhat remarkable old New England family. Among the early settlers of the country, John Porter and his wife, Rose, settled at Windsor, Conn., which he represented in the legislature in 1646. In the fourth generation Colonel Joshua Porter married Abigail Buel, the daughter of Peter Buel, whose wife was the widow of Noah Grant, one of the ancestors of General U. S. Grant. Thus it happens that the great-grandmother of General Grant and of General Barnard was one and the same person.

The boy spent the first twelve years of his life at Sheffield, attending the village school, and was then sent to begin his more advanced studies under his brother, our late colleague, President Barnard, who, after graduating at Yale College, was then teaching school in Hartford. A year later his great-uncle, General Peter B. Porter (for whom Fort Porter, at Black Rock, Buffalo, is named), who was then Secretary of War, offered the boy an appointment at West Point. This was gladly accepted, and he entered in 1829, having just passed his fourteenth birthday and being probably the youngest pupil ever admitted.

He was graduated at the end of the four years' course with second rank in a class of 43 members, several of whom attained

*See frontispiece.

distinction in their subsequent careers. He was assigned to the Corps of Engineers as Brevet Second Lieutenant on July 1, 1833, and, passing through all intermediate grades, became Colonel on December 28, 1865, having received five brevets for distinguished services in the Mexican and the Civil wars. He was retired on January 2, 1881, and died on May 14, 1882. Such in brief is the outline of a career which exerted no small influence upon the current events, both civil and military, of the times in which he lived.

In the civil branches of his profession General Barnard's services covered works of construction and of internal improvement extending from the Great Lakes to the Gulf of Mexico and from the Atlantic to the Pacific coast. In this wide area few important engineering problems engaged the attention of the Government in which his advice was not officially demanded, either individually or as a member of special boards and commissions. His professional duties were not even restricted to the limits of the United States. During the war with Mexico he superintended the construction of defenses at Tampico and made surveys about the City of Mexico, and in 1850 he was named by the President chief of a scientific commission for the survey of the Isthmus of Tehuantepec with a view to establish a route of commerce and travel to our newly acquired Pacific possessions. The report drawn up by J. J. Williams, in 1852, gives the first full account ever published of that isthmus. The exposures incident to this service in the tropics affected his health so seriously that he never entirely recovered. In later life he was sent to Europe twice as a member of commissions to collect information needed by the Government.

General Barnard's military services during the Civil War were conspicuous. At its very outbreak he initiated, as chief engineer of the Department of Washington, field works for the defense of the city. In the Bull Run campaign he served as chief engineer on the staff of General McDowell; and the details of the general plan to turn the enemy's left were established upon his personal reconnaissances, made on the day before the battle, of the route by Studley Springs. At the organization of the Army of the Potomac, in August, 1861, he became its chief engineer, and after greatly extending the defenses of the city he accompanied General McClellan in the spring to the peninsula, and served as his chief engineer during the entire campaign. At the siege of Yorktown he commanded all the engineer troops, and directed the location

and construction of the batteries and approaches. On the Chickahominy he was charged with the construction of the bridges and batteries, and was often consulted as to the position and movements of the troops. After the occupation of Harrisons Landing, contrary to the views of General McClellan, he favored the withdrawal of the army to Washington. On the 16th of August he was individually recalled to that city, and was placed in command of the fortifications,including the troops assigned to their defense; but on the 2d of September he relinquished the latter, not having a rank commensurate with the duty. His commission as Brigadier-General of Volunteers dated from September 23, 1861, while the troops which had become available for defending the city were often commanded by officers of higher rank. He, however, retained the charge of the defenses until their essential completion early in 1864, and was often called upon for reports and advice as to general plans, such as operations against the chief ports of the enemy, the defense of Harpers Ferry, of Pittsburg, of the lake shore, against raids from Confederates in Canada, and also as to important naval problems. In January, 1864, he applied for duty in the field, and on the 5th of June was assigned to General Grant's staff as chief engineer of the armies in the field. He served in that capacity until the surrender of Lee's army, on April 9, 1865, taking an active part in the operations in Virginia. As his office was finally organized, weekly reports of the chief engineers of the two armies, monthly reports of materials received and expended, requisitions for engineer supplies, etc., were submitted to him at General Grant's headquarters. He also devoted much time to careful inspection of the extended lines.

In recognition of his services in the Civil War General Barnard received the brevets of Colonel, Brigadier-General, and Major-General in the Regular Army, and of Major-General in the volunteers. He was also named in the act of March 3, 1865, as one of the one hundred corporators to establish a miltary asylum for disabled volunteers.

Although present in many important battles in the Civil War, and assisting by his counsels in the decision of many important military problems affecting naval operations, as well as those on land, it is upon his services as chief engineer of the defenses of Washington that are based his most enduring claims to remembrance in that crisis of our National history. From a military point of view, the geographical and topographical location of

the capital was unfortunate. It lay within the region where the most important struggles must have place, and it occupied a plain surrounded by commanding heights throughout the greater part of whose circumference an attack might be apprehended if the covering army should experience a serious check. Its loss, even if temporary, must entail disastrous consequences, not only directly upon the conduct of the war, but also indirectly upon our foreign relations, then not always of the most friendly character. These conditions were perceived, but not fully appreciated, in the blind confidence prevailing before the first Manassas campaign; but, after that repulse of the army, the necessity of putting Washington in a condition to be defended by a moderate garrison before the Army of the Potomac could move from its immediate front was seen by all persons of intelligence. Before the first advance, a few field works in the nature of têtes-de-pont had been thrown up to cover the Aqueduct bridge, the Long bridge, and Alexandria, and it is doubtless due to their presence and imperfectly known development that no demonstration was made by the Confederates in front of Washington at the time when demoralization was at its height after the battle of Bull Run. It was fortunate that the duty of extending and perfecting these preliminary works of defense devolved on an engineer so competent to appreciate the problem as was General Barnard. The works of Torres Vedras furnished the best example, but the conditions were so different that a master mind was required for a judicious application of the principles involved. Something more than ordinary field works, but less exacting in time of construction than usual works of permanent defense, was called for; and General Barnard, par excellence, was the man for the occasion. His own monograph, published in 1871 as No. 20 of the Professional Papers of the Corps of Engineers, fully details the semi-permanent system adopted, and will long remain a standard authority on this novel application of the principles of fortification. When completed, the lines enclosed Washington by a cordon of works aggregating 37 miles in length, with sixty-eight enclosed forts and batteries having a perimeter of about 13 miles, actually mounting 807 guns and 98 mortars, with many other emplacements, together with 20 miles of rifle trenches, three blockhouses, and 32 miles of military roads, in addition to those of the District. The utility of these fortifications during the Civil War can hardly be better set forth than in the modest language of their designer:

"When the Army of the Potomac, in 1862, was beaten in the field and to some extent demoralized and disorganized, it fell back on the defenses, where it rested in security; a very few days of respite, the arrival of reinforcements, and a change in the commander enabling it take the field again offensively.

"When Early marched on Washington, in 1864, the defenses had been stripped of the disciplined and instructed artillery regiments (numbering about 18,000 men) which had constituted their garrison, and their places supplied by newly raised 100-days' regiments (Ohio National Guards), insufficient in numbers and quite uninstructed. Under such circumstances much anxiety was felt on the approach of Early's veterans, flushed with recent success, inspired by the very audacity of their enterprise, and incited by the prize before their eyes. Yet, inadequately manned as they were, the fortifications compelled at least a concentration and an arraying of force on the part of the assailants, and thus gave time for the arrival of succor."

Soon after the close of the war General Barnard was made president of the Permanent Board of Engineers for Fortifications and River and Harbor Improvements, a position which he held until his retirement from active service, in January, 1881. The epoch was one of radical transition in coast defense and of vast extension in our works of internal improvement. Our system of masonry coast defenses, to the elaboration of which our former colleague, General Totten, had devoted his life, and which General Barnard has so admirably set forth in his biographical memoir, read before the Academy on January 6, 1866, had been rendered antiquated by the enormous increase in the size and efficiency of heavy guns and by the success in the efforts to mount them on ships protected by armor against shell fire. Our coasts, which had been furnished with fortifications superior to any existing in Europe and in a good state of progress toward completion, were found to be open to attack by a modern fleet of armored battleships. A new type of shipping had appeared just before and during the Civil War, and a new system of coast defense would ultimately be demanded. Pending the necessary studies to determine its character, the existing works must be modified and strengthened to meet immediate needs. The new problem of ordnance and armor was then occupying the attention of the ablest engineers abroad, and General Barnard brought to the study a mind ripened by practical experience in war, a thorough understanding of the

fundamental principles involved, and a technical knowledge of the new developments. Experiments on a large scale were at once inaugurated at Fort Monroe and Fort Delaware by the Engineer Department, and General Barnard, with able coadjutors, was sent to Europe to study the new problems in the light of the most recent investigations there. We have now no occasion to regret either false conclusions or unwise recommendations by the board of which he was so long the president and leading member. During these years he also served as a member of various special boards charged with investigations looking to the improvement of navigation in certain western rivers and at the mouth of the Mississippi, and for a long time was a member of the Light-house Board.

The degree of A. M. was conferred upon General Barnard by the University of Alabama in 1838, and that of LL. D. by Yale College in 1864. He was a member of the American Institute of Architects, and an honorary member of the American Society of Civil Engineers.

Throughout life General Barnard was an untiring student, and he wrote with facility and to the point. Even at the Military Academy he had shown uncommon mathematical ability, and he subsequently carried original investigations in this direction much beyond the limits usually attained by men of so busy a professional career. His papers on the gyroscope and kindred problems, published in *Silliman's Journal* before the Civil War, are examples in point. His writings on technical engineering subjects, both civil and military, were voluminous, and many of them will long remain authorities on the subjects of which they treat. Among them may be mentioned: Notes on Seacoast Defence (1861); Reports of Engineer and Artillery Operations of the Army of the Potomac, prepared with General Barry (1863); Report on the Defences of Washington (1871); Report on the Fabrication of Iron for Defensive Purposes, prepared with Generals Wright and Michie (1871); North Sea Canal of Holland and Improvement of Navigation from Rotterdam to the Sea (1872); Problems of Rotary Motion Presented by the Gyroscope, the Precession of Equinoxes, and the Pendulum (1872). It is, however, in Johnson's Universal Cyclopædia, published in 1874-1877, that the versatility and precision of his mental culture are best shown. He found time to act as one of the associate editors in its preparation, and over ninety scientific and other articles, some of them almost treatises, are from his pen. Among them may be named: Aeronautics; Breakwater; Bridge;

Bull Run, Battle of; Calculus; Gyroscope; Harbor; Imaginaries; Laplace's Coefficients; Light-house Construction; Rotation; Tehuantepec; Variations, Calculus of; and Tides, Theories of. Few engineers have been more profoundly versed in their profession or more able to give reasons for their convictions.

In early life, when stationed in New Orleans, he married Miss Jane Elizabeth Brand, daughter of William Brand and sister of the Rev. William F. Brand, of Maryland, one of the noted clergymen of that State. Four children were born to them, of whom one son survives. In 1860 he married Anna E., daughter of Maj. Henry Hall, of Harford County, Maryland, whose ancestor emigrated with Lord Baltimore; and their three children are all living.

In his personal characteristics General Barnard was a thoughtful, self-contained, and earnest soldier. Under fire he seemed to have no sense of exposure, and in his frequent reconnaissances he was wont to push aside advanced pickets attempting to advise him as to the position of the enemy's sharpshooters, apparently trusting more to his own intuitions than to their local knowledge. His inherited deafness rendered social intercourse somewhat difficult, and to those who did not know him intimately this circumstance perhaps conveyed the idea of coldness and formality; but such was far from being his nature. As his aide-de-camp during the Peninsular campaign, I often saw evidences of the warm interest he took in the success of the many young officers serving under his orders and of his cordial appreciation of good work done by them. His brother, our late colleague, being at the South at the outbreak of hostilities, had experienced no little difficulty in crossing the line or communicating with his family, and when he suddenly appeared unannounced in the General's tent one evening, the fraternal embrace which followed proved that they both concealed warm hearts under a dignified exterior. He had a keen sense of humor and a passionate love of music. Indeed, he composed many pieces—among others, a *Te Deum* that still survives.

General Barnard was nominated by the President, on the death of General Totten, to succeed him as Brigadier-General and Chief of Engineers, April 22, 1864; but the nomination was withdrawn, at the request of General Barnard, before any action was taken by the Senate. He never, to my knowledge, made public his reasons for this request; but the facts suggest the fair inference not only that President Lincoln highly appreciated the merit of General Barnard's services, but also that the latter's sense of justice to a

superior in rank in the Corps of Engineers forbade him to take advantage of this appreciation. Such self-sacrifice is not common, either in or outside of the Army.

The estimation in which General Barnard was held in the Corps of Engineers is well expressed in the concluding paragraphs of the order of General Wright announcing his death:

"A service of nearly fifty years in the Corps of Engineers has been closed by the death of one of the most prominent of its members.

"Of greatly varied intellectual capacity, of a very high order of scientific attainments, considerate and cautious, ripe in experience, sound in judgment, General Barnard has executed the important duties with which he has been charged during his long and useful life with conscientious care and regard for the public interests and with an enthusiastic devotion to his profession. His Corps, the Army, and the Country are his debtors.

"Modest and retiring in disposition, considerate and courteous, warm in his sympathies and affections, our deceased associate will be missed as few are missed, and his name, which will be held as one of the foremost names of the Corps of Engineers, will be cherished with peculiar love and affection by his brother officers."

River and Harbor Notes from Foreign Lands

Coast Erosion and Protection.*

BY

Mr. ERNEST R. MATTHEWS
A.M. Inst. C.E., F.R.S. (Ed.), F.R.G.S., F.G.S.,
Borough Engineer of Bridlington

WAVE ACTION.

(*a*) *Laws Relating to Wave Action.*—Waves are of two main classes: tidal and wind waves. It is not the intention of the author to describe the laws which govern wave-action, as the subject is too large a one to be dealt with in this article. The nature and action of waves have been from time to time the subject of much investigation and experiment, the most important experiments carried out being undoubtedly those of the late Astronomer Royal, Sir G. B. Airy; and those of Messrs. Weber, Robertson, and Scott Russell. The results of the Weber Brothers' researches were published at Leipzic in 1825, and at the meeting of the British Association at York in 1844 Mr. Scott Russell reported on his researches, the title of his paper being "On Varieties, Phenomena, and Laws of Waves, and the Conditions which Effect their Genesis and Propagation." Sir G. B. Airy's experiments are set out in his treatise on "Tides and Waves."

The velocity and height of waves, and the laws generally which govern wave-action, may be passed over. There are two matters, however, which must be considered in designing sea-walls or breakwaters, *e. g.*:

1. The force of impact of a wave.
2. Height to which a breaking wave will rise.

(*b*) *Impact of Waves.*—Some of the forces of Nature can be measured, such, for example, as the velocity of the wind, the pressure of water due to a given head, and the rate of erosion on a coast line; but Smeaton was correct when, in referring to certain other powers of Nature, he said: "They are subject to no calculation." Thus it is that the marine engineer's task is often a difficult one. The force with which a wave strikes a plane surface, while it may be gauged approximately, can not be accurately measured; for while a marine dynamometer will record a certain blow, this

*Reprinted, by permission, from *Engineering*, August 30, 1912.

really does not represent the force of impact of the wave against, say, the upright face of the sea-wall to which the instrument is fixed, for the wall is non-elastic, while the dynamometer is usually fitted with springs or plungers, and is therefore an elastic body.

The force of impact of a wave will depend upon five things:

1. The nature of the wave (whether a tidal or a wind wave).
2. Its length and height.
3. Its velocity.
4. The nature of the body against which the wave strikes—whether elastic or rigid.
5. The shape and position of such a body—whether it has a vertical or sloping face.

The greatest recorded force of impact of a wave was obtained at Dunbar, in the county of East Lothian, when a force of 3½ tons per square foot on the surface was registered. A force of 3 tons per square foot has been registered frequently. There can be no doubt that higher results than these may be obtained. The displacement of heavy bodies, and the height to which a wave rises after striking a vertical face, would indicate that the record given by the dynamometer is less than the actual force exerted. Let these two points be considered for a moment.

The Displacement of Heavy Bodies by a Wave.—This is well illustrated during every severe storm by the washing away of a portion of a sea-wall and promenade on some part of our coast, or by other damage done. During the construction by the author of the Parade Extension Works at Bridlington, for example, a number of blocks of stone, each weighing from 3 to 4 tons, were washed out of position and carried several yards away. This occurred during a severe north-westerly gale, and a view of the structure so damaged is given in Fig. 1 (p. 93).

"At Cherbourg breakwater upwards of 200 blocks of concrete, each weighing 4 tons, were lifted by the waves during a north-easterly gale and taken over the top of the mound and deposited inside. Blocks of 12 tons were moved from their place and turned upside down."*

During a storm at Wick huge blocks of concrete, weighing 1,350 and 2,500 tons, respectively, were displaced. This seems scarcely credible, and doubt has been raised as to whether this movement was due entirely to wave action; it would seem to be within the bounds of possibility, however, when we consider that in 1894, during a storm, a large portion of the breakwater at the Bilbao Harbor, estimated to weigh 1,700 tons, was overturned, due entirely to wave action. The old Beaconsfield sea-wall at Bridlington was entirely demolished by a storm, or series of storms, which occurred about eighteen years ago, and huge pieces of the wall of concrete and brick, weighing 20 tons or more, are now to be seen upon the foreshore. During the storm which occurred in No-

*"Tides and Waves," by W. H. Wheeler, M. Inst. C. E.

vember, 1911, much damage was done by the sea at Heysham, large pieces of the sea-wall being washed away.

Other examples might be given, but these will doubtless suffice to show that the force indicated by dynamometers is really less than the force actually exerted. It should also be noted that when a wave strikes a body, its action is continuous as long as the wave lasts, and not momentary, as when a solid strikes a solid. It is this continuity of impact that causes the damage.* Mr. Thomas

Spurr, Bridlington, Photographer

Fig. 1 (top). New Victoria Sea Defences, Bridlington.
Fig. 2 (bottom). Wave Breaking Against Sea Wall at Bridlington.

Stevenson puts it in a simple way when he likens it to "a continuous succession of cannon balls falling on the body."

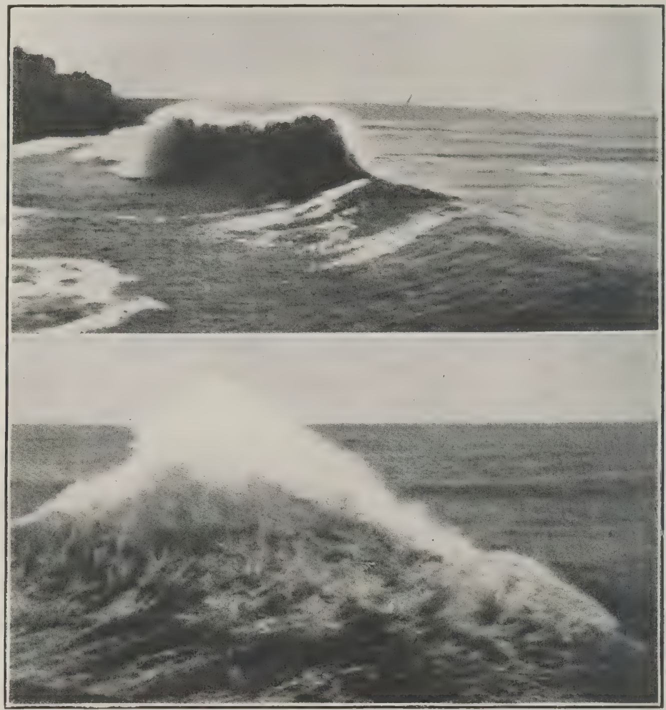
"The Ymuiden breakwaters (Holland)† are vertical, or nearly vertical, structures, with mounds of concrete blocks, termed 'wave

*"The Design and Construction of Harbours," by Thomas Stevenson, F. R. S. E.

†"Principles and Practice of Harbour Construction," by William Shield, F. R. S. E.

breakers,' on the seaward side. These mounds are composed of blocks weighing from 5 to 10 tons each, thrown pell-mell into the sea. The mound has a slope of about 1½ horizontal to 1 vertical, and is surmounted by a row of 20-ton blocks. During a gale one of these 20-ton blocks was lifted by a wave to a height of 12 feet (vertically up the face of the pier) and landed upon the top of the pier, which was 4 feet 10 inches above high water.''

The most destructive seas usually occur during an off-shore gale, a heavy ground swell is then propagated; this is illustrated in the



Spurr, Bridlington, Photographer

Fig. 3 (top). Breaking Wave at Bridlington.
Fig. 4 (bottom). Back-wash Meeting Incoming Wave.

diagram, Fig. 5 (p. 95), and in the view, Fig. 3 (p. 94), the latter showing such a wave on the point of breaking. In this case the wind has dropped somewhat, but the ground swell remains. Fig. 2 (p. 93) illustrates the effect after such a wave has broken, and represents a rough sea at Bridlington, while Fig. 4 (p. 94) shows the back-wash of such a wave meeting the incoming wave. While waves usually assume their greatest force during an off-shore gale, it nevertheless is a fact that in such a gale beaches invariably heap up, while during an on-shore gale the foreshore is depleted. The

author's experience, however, has taught him that serious damage to a sea-wall, or other form of sea-defence running parallel to the coast-line, seldom occurs during an on-shore gale; there is too much broken water during such a gale for much damage to be done. The type of wave propagated at such a time is shown in the diagram (fig. 6, p. 95) and in the views (Figs. 7 and 8, p. 96).

The Height to Which a Wave Rises After Impact.—This is a very interesting matter. The following reply was recently given by the author to a query on the subject raised by one of the members

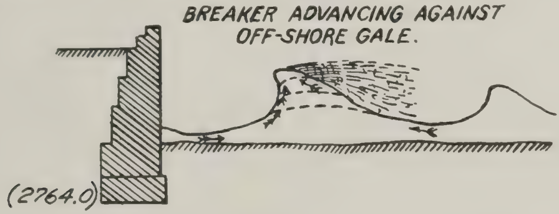


Fig. 5.

of the Society of Engineers (see Transactions of the Society for November, 1911):

"Mr. E. R. Matthews, A. M. Inst. C. E., F. G. S. (Member), borough engineer of Bridlington, writes as follows with reference to the inquiry which appeared in our last issue:

"'It is quite a common thing for wave action to take effect to a height of 30 or 40 feet above mean sea-level. In the case of a vertical cliff it would very seldom take effect at such a height, but

Fig. 6.

where the face of the cliff slopes, through landslips or other causes, action does frequently take place to the height named. I have observed that on the Holderness coast of Yorkshire, on cliffs 30 to 35 feet in height, seaweed has been washed on to the top of the cliff. In this case, high water of ordinary spring tides would be about 7 feet above the base of the cliff. In the case of a sea-wall the wave action is more pronounced, and where there is no projecting cornice to throw back the wave I have frequently observed that waves striking a sea-wall, which is, say, 35 feet in height, are carried almost as high again as the height of the wall (see fig. 9) before they fall on to the promenade at the back of such wall.

"'The best example that I know of where this action frequently takes place is the Marine Drive Sea-Wall at Scarborough. The height of this wall is about 35 feet above the level of the foreshore, and high water of ordinary spring tides reaches, I should say, about 15 feet up the wall. Waves striking this wall frequently rise to a height of about 30 feet above the promenade level. The new sea-wall recently erected by me at Bridlington, in connection with the extension of the Royal Prince's Parade, is 28 feet in height above the level of the foreshore, and has a stepped face

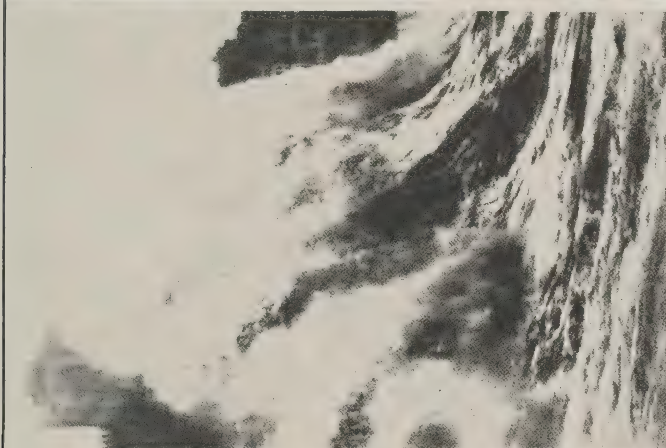
Spurr, Bridlington, Photographer

Figs. 7 and 8. Type of Wave from On-Shore Gale.

(stone), the depth of water at ordinary spring tides being about 10 feet, and I have frequently observed that, in spite of the projecting cornice, a wave will occasionally rise to a height of 20 feet above the top of the wall.'" In Shield's "Harbour Construction," page 81, we read: "It is no uncommon occurrence for storm waves, striking a vertical breakwater face, to throw heavy masses of water to a height of at least 100 feet, often very much higher. Such water in its descent, on reaching the roadway of the breakwater upon which it falls, will have attained a velocity of about 80

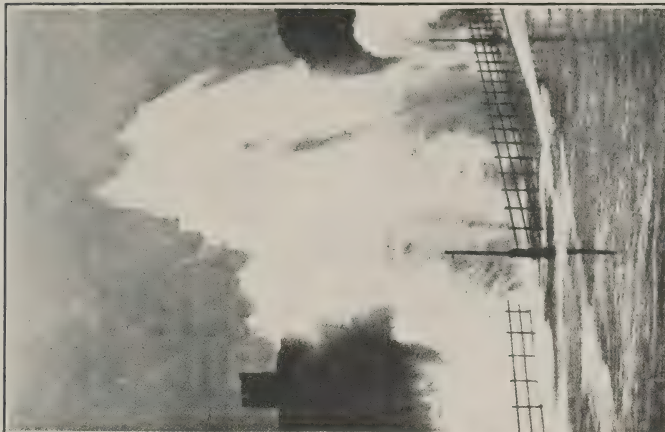
Spurr, Bridlington, Photographer

Fig. 9. Wave at Bridlington.



Foster, Hastings, Photographer

Fig. 10. Wave at Hastings.



Cromack, Scarborough, Photographer

Fig. 11. Wave at Scarborough.

feet per second, or nearly double the velocity and four times the force of the water striking the face of the breakwater.''*

Thomas Stevenson, in "The Design and Construction of Harbours," also gives some interesting information on this subject. The force with which a wave strikes a sea-wall, and the height to which such wave rises after impact, are well illustrated in fig. 10, page 97, which represents a wave striking the Baths sea-wall at Hastings during a storm on October 22, 1911, and in fig. 11, page 97, which is a view of a huge wave rising after impact with the sea-wall on the north shore at Scarborough.

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*"Principles and Practice of Harbor Construction," by William Shield, F. R. S. E.

Oranization of the Services of Public Works in France*

Translated by

Maj. F. A. MAHAN
Corps of Engineers (Retired).

Although the term "public works" is found in the text of many laws and especially in the law of Pluviôse 28, year VIII (February 17, 1800), no explicit definition thereof was given anywhere by the legislator, and resort to jurisprudence was necessary to determine the boundaries which divide the works called public from the works called private. It may be said, in general terms, that public works are those carried on in the interest of the public services of which a great State has charge. So it is that not only the works performed for the services of the Ponts et Chaussées, Mines, Navigation, and Railways are public works, but also that the works carried out by the Departments, Communes, Syndical Associations, the various Ministries are themselves public works so long as their object is the public weal or a somewhat extended collective interest. The detailed study of these works would be long, diffuse, and difficult. Furthermore, it has already been made in a masterly way, from the juridical point of view, by Mr. Albert Christophle, former director of the Crédit Foncier of France, and the reader, who, after having read what is here given, desires to study this delicate subject thoroughly, is referred to his detailed volume. The sole end of the present modest book is to sum up rapidly the present organization of our various services of public works by recalling, in each one, the most salient facts of its history and by giving the principle modes of action of each one.

The history of our public works, from Charlemagne to the present time, is a striking example of rapidly increasing administrative centralization. As these works developed and as they increased in numbers and importance, they were gathered by Royal authority under the direction of an Administration which became more and more numerous, more and more highly educated, and more and

*Translation of "Organisation des Services de Travaux Publics en France," by M. Campredon. Brought up to 1912 by the translator.

more powerful. The creation of the office of Grand Voyer* of France in 1599, the formation of the General Direction of the Ponts et Chaussées (Bridges and Roads) in 1715, the foundation of the School of the Ponts et Chaussées in 1745, the reorganization of the Corps of the Ponts et Chaussées in 1790, the creation of the Department of Public Works in 1830, and its reorganization in 1839, are so many steps of this gradual advance which was to end in the gigantic Administration of the third Republic, now made up of 10,000 functionaries. It is necessary, however, to point out that the tendency toward decentralization which appeared at the time of the law of May 10, 1836, has become more and more marked since that time (laws of 1866, 1871, 1884) up to the project for unifying the road services now (1895) submitted to the Parliament. This decentralization, which has thrown on the departments and communes a part of the work which until then was cared for by the State, and which tends now to give them a still greater share in the maintenance of the great lines of communication, marks a retrogression on the general movement which had presided over the organization of our public works, and had made the year 1839 a sort of maximum in this curve of the land roads which, after having been so long on the increase, is now more and more on the decline.

The Ministry (or Department) of Public Works, in which are now centralized all the great services of general interest, includes at present, outside of the Office of the Minister and the Directions of the Personnel and of Accounts, two main divisions:

1. That of Roads, Navigation and Mines;
2. That of Railways.

The service of Civil Buildings and National Palaces, which had been for several years a part of the Department of Public Works, was transferred in 1895 to the service of the Fine Arts, belonging to the Department of Public Instruction. This classification will be followed substantially as this book goes on. The importance of the first service, however, makes it necessary to divide it into two studies: That of the Ponts et Chaussées and that of the Service of

*Voyer is an officer who has charge of the construction and police of roads and streets.

The Duke of Sully (1559-1641) was the first minister, after the foundation of the monarchy, to realize the great importance of the roads for the internal commerce of the kingdom; he created the office of Grand Voyer or, as we might say, General Supervisor of Roads, and became himself the first incumbent.

Mines, services which are wholly distinct from the double point of the way in which the personnel is obtained and of the nature of their works. In like manner, the relative unimportance of the Direction of Civil Buildings caused its being placed in the series of Auxiliary Services in the last part of this volume. Finally, a great deal of room has been given to the Services of Collective Interest, which are tending toward a constantly growing importance in our modern organization, in accordance with the rising development and strengthening of the personality of the Departments, Communes, Syndical Associations, Public Establishments, Chambers of Commerce, Companies and Societies having a moral personality. Hence this study can be summed up in the following programme:

First Section.—Services of general interest—

First part.—Service of the Ponts et Chaussées;

Second part.—Service of Mines;

Third part.—Service of Railways.

Second Section.—Services of collective interest—

Fourth part.—1. Departmental service;

2. Communal service;

3. Service of syndical associations;

4. Service of other works of public utility.

Third Section.—Auxiliary services—

Fifth part.—1. Colonial service;

2. Service of the Ministry of Public Instruction, Fine Arts, and Worship;

3. Service of the Ministry of Agriculture;

4. Services of the Ministries of War and of the Navy;

5. Service of the Ministry of the Interior;

6. Service of the Ministry of Commerce, Industry, and Posts and Telegraphs;*

7. Service of the Ministry of Finance.

The study of each of these services will be subdivided into three principal parts:

1. Historical study;
2. Present organization of the personnel;
3. Mode of procedure and maintenance of the offices.

Every endeavor will be made to have this study as interesting and as nearly up-to-date as possible by dwelling on the Special

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*The postal, telegraph, and telephone services were transferred from the Ministry of Commerce and Industry to that of Public Works.

services, such as the Colonial Service and the Service of the City of Paris, which are becoming more and more important, and by pointing out reforms which, in many of the services, are under consideration. We shall be happy if we succeed in making this study sufficiently exact to reproduce the general physiognomy of the present organization of the services of Public Works and sufficiently interesting to be read without fatigue by all those who may be desirous to examine the subject.

The author does not wish to close these few introductory lines without sincerely thanking all those who have been interested in this study and who have helped him with their knowledge and their advice. Members of the editing and patronage committees of the series of works of which this is a part, chiefs of bureau in the various ministries, all have a right to his profound gratitude, for they have spared neither their time nor their kindness, and he is glad to be able to make publicly this acknowledgment of the share of his modest efforts which is their due.

FIRST PART.

SERVICE OF THE PONTS ET CHAUSSEES.

HISTORICAL STUDY.

The great military roads which the Romans had spread all over Gaul fell into ruins at the time of the invasion of the Barbarians, and the earlier of the Frankish kings, occupied solely in making war, do not seem to have been sufficiently civilized to understand the importance of lines of communication. The only conjecture remaining of this distant time is the tradition expressed by the term of "Brunehault roadways" (chaussées de Brunehault), which appears to have been given to old Roman paved roads, still intact vestiges of a civilization which has disappeared. The time of Charlemagne must be reached in order to find in the Capitularies the trace of thought and arrangements relating to lines of communication. But the feebleness of power, under Charlemagne's successors, scarcely allows any belief in the efficacy of these prescriptions, and feudal anarchy has left no written testimony as to the condition and maintenance of public highways during the Tenth, Eleventh, and Twelfth centuries. But, in the reign of Philip Augustus, at the end of the Twelfth Century, the movements of peoples started by the crusades, the development of ideas, the freedom of the commons, and the strengthening of royalty

gave a certain rise to commerce, to industry, to the relations between the various provinces, and the reestablishment of lines of communication was begun, especially the reconstruction of bridges, roads, and "passages."* The constructions were undertaken, for the most part, by religious orders, of which the most celebrated was the Order of the Pontit Brothers, made illustrious by St. Benezet, the builder of the bridge at Avignon (1177-1185). During this period of transition, when civilization and science, emerging from the shadows of the middle ages, seem to have been preserved in the latent state by the ecclesiastical world and in the shade of the cloisters the monks were the principal engineers. Thus, for example, the Notre Dame bridge at Paris was built in 1500 by a Franciscan monk, Brother Jacondus; the boat bridge at Rouen, by an Augustin, Brother Nicolas; the bridge at Maëstricht and the Tuileries bridge, by a Dominican, Brother Romain. The funds consisted mainly of tolls for crossing bridges or for use of the roads and for bait, and secondarily of pious gifts, small sums collected on transactions of various sorts, slight subsidies from the royal treasury and contributions from the salt tax and other funds collected for the State. Unfortunately, most of these funds, and especially the highway tolls, were often turned aside from the purposes for which they were collected by lords of the land who were rebellious to royal authority, and the royal edicts which have come down to the present time are filled with prescriptions relating to the collection of highway tolls and their proper use. Thus the edict issued in September, 1535, at Fontaine Française, by Francis I, said: "Monies paid for tolls have been taken and collected by vassals who hold them by grant from us and from our predecessors or by long-standing or immemorial possession, as revenue belonging to them and as forming part of their fief and seigniory, without making any repairs; now wishing that the monies which we and our predecessors have allowed to be gathered and raised be used as they should be and not otherwise, we being moved to this for these causes and other good and just occasions, . . . say, declare and order, will and it so pleases us, of our own motion, sure knowledge, full power and royal authority, that all and every of the monies of the tolls aforesaid . . . be employed respectively for the repairs of bridges, roads, ferries and paths of places and defiles at which the said tolls are taken, so that people can pass there, go and come in safety, without danger, inconvenience and

* A "passage" is an unclassified roadway opened by private individuals who are bound to keep it up.

injury to persons, mounts, provisions, merchandise, and other goods.'' It was these same abuses which the famous ordinance of Charles IX (January, 1560) prepared by the Chancellor of the Hospital, sought to reach. Article 105 of this ordinance contained the following: ''Those to whom the right of tolls belongs will be required to keep, in good and due repair, the bridges, roads and ferries; otherwise, failing to do this, we enjoin our legal authorities to sieze and place in our hands the revenues from the said rights and to cause these to be used for the necessary repairs. And where this may not suffice, to take again the monies from those who have received the tolls up to what does meet the cost of the said repairs.'' But the number, repetition and succession of these edicts are flagrant proofs of their inefficacy. Funds continued to be misused, the works to fall into ruin, the roads to be neglected, and we must come down to the time of Colbert to see an end put to this waste and abuse.

However, from the beginning of the Sixteenth Century, with Henry IV and Sully, the firmer royal authority began to take up the organization of a service of public works, and in 1599 was created the office of Grand Voyer of France, whose principal duties were to keep himself informed as to all the tolls levied for public works, by virtue of royal commission, and to visit all works done or to be done. He caused himself to be assisted in his visits and duties by assistants whom he appointed in each district and by the intendants and treasurers of these districts. But this small personnel, who had also other important financial or administrative functions to which to attend, did not render him good service, and most of the works still escaped the control of the Grand Voyer. All the same, great progress was made during this reign, especially the introduction of bids as the general rule for carrying on public works, with the greater part of the guarantees called for by our present specifications and the constantly increasing contributions of the royal exchequer to the cost of public works. In 1609, this contribution reached the amount of 870,000 livres (very nearly $174,000) which was devoted almost exclusively to the improvement of navigable highways. Among the great works performed at this time must be mentioned, in the first place, the beginning of the Briare Canal, on which Sully and 6,000 soldiers were employed, and in the second place, the drainage of the marshes of the whole kingdom grand to the Dutchman Bradley. Must be mentioned among the works of the second class all those carried on for

the purpose of making a large number of the rivers of the kingdom navigable, especially the Loire, the Seine, the Aisne, the Vesle, the Vienne, and the Clain. But the troubles which followed on the assassination of Henry IV and the weakening of the royal power which was its result, interrupted the growing administrative organization of public works. The edict of 1621, which left with the treasurers of France the disposition to be made of tolls on bridges and roads, and the edict of 1626, which suppressed the office of Grand Voyer, caused the public works to fall back into the hands of the districts, and their care again became provincial and independent. The treasurers who had charge of these works neglected them more and more, and the great highways, bridges, roads and ferries fell into a condition of complete decay. The river lines were less neglected, however, but perpetual concessions to private parties or to private industries took the place of works carried on directly by royal authority and under its control. Letters patent gave, in 1638, a grant to Messrs. William Bouteroue and James Guyon for the Loing Canal; in 1643, the navigation of the Ardêche to the Marchioness of Montlor; in 1644, the canal from the Rhône to the port of Agde, to Mr. James Brun; in 1655, the canalization of the rivers of Champagne to Messrs. Hector Bouteroue and Peter Barillot. This system of perpetual concessions to insure the onerous carrying on of public works by private parties was a matter of common occurrence; furthermore, the old form of government found in it a convenient way for not taxing the royal exchequer and for throwing off on others the responsibility involved in large works of art.

The royal authority, which had become firmly seated and all powerful under Louis XIV and Colbert, thought seriously of repairing the lines of communication and of reorganizing the Service of Public Works. The great minister, who was the first to hold the title of Controller General of the Finances, considered the development and improvement of public highways as one of the first needs of trade and one of the most fertile sources of public prosperity. Consequently, he devoted to it his greatest care and created intendants in each district who had, among their duties, the administrative and technical part of the roads and bridges, and left with the offices of the treasurers of France only the financial and legal part. These intendants corresponded directly with him, were responsible to him for their services and were assisted by "*trésoriers commissaires*," selected from among the treasurers of

the district offices, at the rate of one per district. The duties of these *trésoriers commissaires* were, as laid down by a decision of the Council of State, dated February 11, 1681, "to visit with the intendant of the district or by himself, as the said intendant shall judge better, the bridges and roads of the said district, prepare a report of their condition and have prepared in his presence, by intelligent and capable persons, specifications and estimates of cost of the work necessary to repair them and to keep them in good order; after which, in connection with the said intendant and not otherwise, shall be taken up the award after bids of the said works, the publicity required in such cases having been previously given; and after they shall have been completed, they shall be received in the usual way."

So it is seen that the duties of these *trésoriers commissaires* were somewhat analogous to those of the assistant engineers of the Ponts et Chaussées and that those of the intendants were not unlike those of the chief engineers. What remains of the correspondence of Colbert and the high pay granted to these functionaries show how important their duties were considered. Furthermore, by reason of this skeleton of functionaries and the indefatigable and intelligent activity of their first minister, the public works of France took a new start. Tolls continued to be the main source of income for public works, but the abuses to which they had given rise up to that time were severely repressed, and the regulations promulgated by the royal declaration of January 31, 1663, laid down rigorous penalties for all those who should not use the rights of toll for the maintenance or repair of the bridges, roads and ferries which had been granted to them. The other resources for public works were allotments, always too small, from the royal exchequer which formed, after the reign of Louis XIV the *état-du-roy* for bridges and roads, contributions furnished by the cities, districts, or provinces which were interested in the lines of communication to be repaired, maintained, or constructed. This share of the cities became constantly greater as the road system was perfected, and came from their *octrois** or from temporary *octrois* laid for that purpose.

To those various sources of income were added the personal service tax laid on the frontier provinces newly acquired by conquest, Artois, Lorraine, Alsace, Franche-Comté, Dauphiny, and used mainly for making over the main roads. But Colbert, who under-

*The *octroi* is a sort of customs duty levied by cities on provisions, building materials, fuel, etc., coming into them.

stood all the abuses which might grow up under the system of the personal service tax, never allowed it to become a general measure even in those frontier countries where alone it was applied. It was extended to the whole kingdom only half a century later, and it became further on one of the main grievances of the people against the old régime. By means of this cooperation of money and arms, the great highways were repaired, widened, paved and multiplied; large bridges were put in order or built; the navigation of the Loire was improved and the land along the river was protected against overflow; the floods of the Drac were held in bounds by levees; the Oise, the Aube, the Seine, the Marne, the Somme, the Guyenne, the Lot, the Tarn, the Dordogne, the Isle, the Vézère were canalized and made navigable. Among the great works of this period must be mentioned the canal from Calais to Saint Omer (1681-1683), the Loing Canal (1679-1692), and, above all, the Languedoc Canal, which remains one of the most glorious works of the time of Louis XIV, and which, by the genius, perseverence, and patriotism of Peter Paul Riquet, was built during the fifteen years from 1666 to 1681. Other projects for great canals were taken under consideration, especially the canal from the Lower Rhône to the sea, from the Saône to the Loire by way of the Charolais*; from the Saône to the Yonne by way of Burgundy. These projects, however, remained in suspense on the death of the great king, and most of them were not to be carried into effect until our own times.

Colbert's great work and the organization with which he had endowed the service of Public Works were, unfortunately, not destined to be perpetuated after him. In the midst of the wars and political difficulties which darkened the last days of the reign of Louis XIV, the great roads and bridges were abandoned to their fate; the abuses reappeared. The greater part of the resources devoted to public works were turned over to the war budget, which was more urgent, and there are to be recorded few decisions during the times of the Controllers General of the Finances, Chamillart and Desmaretz, favorable to the development of public works. However, the decision of May 26, 1705, must be noted, as it laid down the principle of building roads in a straight line and laid the foundation for the right of exercise of eminent domain in matters of public utility. An attempt made in 1712 to reorganize

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*The Charolais is a small district near Charolle in the southern part of the old Province of Burgundy.

the personnel of the Ponts et Chaussées must also be mentioned. Colbert had called upon certain men, in each district, persons skilled in the arts of construction, architects, masons, and carpenters, to prepare specifications and plans of works to be carried out under the orders of the intendants. Some, by their greater knowledge, were not long in being charged exclusively with the technical part of the works and received the appointment of *architects to the king*. These engineer architects, few in number for the entire kingdom, were finally attached, in 1712, to the service of the king, they being paid from the royal exchequer, and their number was increased to twenty-two, one for each district. Moreover, a decision of November 28, 1713, appointed eleven inspectors general, who were to inspect them every year, in accordance with orders and instructions from the Controller General of the Finances, and to prepare everything which they might deem necessary for the re-establishment and maintenance of the highways, bridges, roads, and other public works. But this organization, similar to the one of to-day, was in operation for only two years, when the resources of the exchequer became such that it could not be continued. The engineer's art does not seem yet to have made any great advance at this time, and the engineers who were called on to lay out and carry on great works were, first of all, architects, and better adapted to build palaces than to construct roads and bridges. The most celebrated of them, Hardouin Mansart, who had charge of rebuilding the Moulins bridge across the Allier, spent more than a million and a half (francs) on this work without obtaining a solid construction, as the bridge was carried away, before it was fairly finished, by the first flood of the river. It was finally rebuilt, for the third time, only in 1752. Still, some great works were built by the system of concessions before the end of the reign of Louis XIV. There should be mentioned especially the concession of the navigation of the Loire, between Roanne and Saint Rambert, to the Sieur de la Gardette in 1701; that of the navigation of the Eure, between Chartres and Pont de l'Arche, to Madame de Maintenon in 1704; that of the navigation of the Clain to Madame Marchand de la Mulinère in 1708.

Adjudication* also continued to be adopted for many large works which were paid for by special taxes laid on the regions which were to profit by these works. Thus were built by adjudica-

*Adjudication, while differing in application from an American system of bids, is founded on the same principle of competition.—F. A. M.

tion the canal works in Languedoc and Provence (the Losnes Canal in 1711) and the levee works of the upper basin of the Loire (the Pinay and La Roche levees in 1711-1717). But the general condition of the routes and roads of the kingdom was deplorable, as is shown by the *Memoir for perfecting the police of roads* by the Abbé de Saint Pierre. It was impossible to travel on them in winter, and the rows of trees which bordered them, regularly cut by the adjoining farmers, added the invasion of their branches to the constant humidity, of which they were the seat, for lack of a regular primitive construction.

The reorganization of the personnel of the Ponts et Chaussées reappeared, under the Regency, with the institution of a General Direction of the Ponts et Chaussées, of which the first person to hold the place was the Marquis of Beringhen. This office, which centralized everything belonging to the Service of Public Works, does not appear to have been made independent of the Administration of the Finances, which continued to make the appointments in the personnel of the engineers and intendants and to countersign all decrees relating to the Service of Public Works. After the Marquis of Beringhen, it fell to Joseph Dubois, brother of the Cardinal, one of the secretaries of the cabinet of the king, and when, in 1736, the latter asked permission to give up his duties on account of his great age, the office was suppressed and its duties were transferred to Ormesson, Intendant of the Finances, who received as his commission "the detail of the Ponts et Chaussées, the paved streets of Paris, levees (*turcies et levées*) beaconage of the Loire and rivers and tributaries, both for finance and for their full and entire administration."

Nevertheless, in spite of this subordination of the Director of Bridges and Roads to the Intendant of Finances, the first organization of the Corps of the Ponts et Chaussées dates from this time (1716). There were substituted for the eleven inspectors general and the twenty-two district engineers, one inspector general, one architect first engineer, three inspectors and twenty-one engineers. This hierarchical formation lasted through the whole of the Eighteenth Century, nothing being said of the increases called for from time to time by the growing needs of the service. Among these first engineers must be mentioned Trésaguet, who was to inaugurate the maintenance of the main roads surfaced with stone and who was charged especially with rebuilding the Moulins bridge. Outside of this framework of the Ponts et Chaussées, other engi-

neers were appointed especially for special services, like the two de Regemorte, father and son, one of whom had charge of the works of the Orleans Canal and of the bridge at Blois, and the other of putting in order all the main highways of the province of Alsace. Another engineer of this family, which gave not less than four great engineers to France, had charge of the levee service of the Loire. But the salaries of these poor engineers of the king, which did not exceed 1,800 livres a year, were often neglected and remained unpaid for long years. Decisions of the Council of State raised several times these all too small salaries, but the Chamber of Accounts at Paris, which was hostile to all functionaries who did not hold a title to offices which were bought or inherited, refused to take account of these decisions. It was only after repeated complaints, which have been preserved in letters of the time, that the salaries of the engineers were raised at last to 2,400 livres and were paid regularly as the result of the letters-patent of July, 1733. Some, like Trésaguet, were retired with a pension as the result of long years of service. This hierarchical organization of the Ponts et Chaussées did away, nearly everywhere, with the important part which the intendants had held under Colbert, and they only continued to exist through respect for formalities and custom. An exception was made, however, until 1720 for the district of Paris where the three treasurers of France were assigned to the service of the Ponts et Chaussées. After 1720, the Government, which saw in these prerogatives of the Bureau of Finances of Paris an encroachment on its own initiative, appointed four intendants to take charge of all works in the district of Paris. Through the organization of the Service of the Ponts et Chaussées, the public works underwent a sensible improvement during the whole of this period, in spite of the inextricable difficulties in which the public finances were floundering at this time, which was made celebrated by Law's system and its consequences. Streets were paved and highways were smoothed down and widened (decrees of May 3, 1720, and of June 17, 1721).

The road police began to appear with the royal declaration of November 14, 1724, forbidding more than four horses to be hitched to any two-wheeled cart from October 1 to April 1, or more than three horses from April 1 to October. Larger sums were devoted to public works, these sums being obtained by extraordinary taxes laid on the different districts, and closer oversight caused abuses and malversations to disappear.

Most of the large bridges were built or made over; bridge at Charenton (1714-1719); bridge at Blois (1716-1723); bridge at Beaugency (1718-1725); bridge at Saumur (1717-1730); bridge of la Charité (1724-1731); Saint Maxence bridge (1717-1720); bridge of la Guillotière (1718-1724); bridge at Bray sur Seine (1715-1730); bridge at Compiègne (1725-1730). Many of these bridges, made of wood, had to be rebuilt later, but they show already a much more perfect knowledge of the art of the engineer and a much greater activity in carrying on public works. Among the great works constructed for the improvement of navigation, must be mentioned the opening of the Orleans Canal, in 1724, the construction of which had been directed by Jean Baptiste de Regemorte; that of the Crozat Canal between Chauny and Saint Quentin, in 1733; and the resurrection of the project for the Burgundy Canal, in 1727.

We come now, in this rapid historical sketch of the public works in France, to the sadly memorable date of June 13, 1738, when an order, signed by the Controller General of Finances, Orry, established throughout France personal service, or service in kind on the main roads. This system of personal service, which was to last for fifty years and which remains as one of the greatest blunders of the old régime and one of the principal causes of its unpopularity, had been introduced gradually; at first temporarily, when Louis XIV travelled through the kingdom, then for special works of interest to abutting property owners, finally in a permanent and uniform way in the eastern provinces which had recently been acquired by conquest. But it only became a general measure when the General Direction of the Ponts et Chaussées, having been suppressed, fell into the hands of the Department of the Finances.

Controller General Orry, who had been intendant at Metz and had noted there the great services which the service in kind had rendered to the road system in that country, decided to extend it throughout France. But, as a matter of fact, it was applied very unequally and was subjected to the good pleasure of the Intendants of the Finances. For example: whereas only six days of personal service were called for in a year in the Soissons district, in another district, according to what is related by Richer d'Aube in his *Memoir on the functions and duties of intendants*, there was no fear as to keeping at work 51,000 men, with a proportionate number of pack and draught animals, during thirty-nine days in the year. The anonymous document still in existence and called:

Memoir on the execution of work by personal service sought to make the service uniform and to lay down its essential regulations. An idea can be formed of the state of mind which animated those in power at this time from these few sentences taken from the midst of the jumble of instructions and recommendations: "Children of both sexes more than twelve years old may be called upon to carry pebbles and sand from the piles to the laborers on the works . . . The inhabitants of the villages nearest the roads will be required to give shelter to the workmen who are summoned and who cannot return home, and to furnish bedding of fresh straw for the men and litter for the animals . . . Those who are liable for personal service are forbidden to hire substitutes, and subdelegates have full authority to imprison or to punish with fine and garrison* those liable and not presenting themselves for this duty."

In spite of this memoir and offical instructions from Controller General Orry, the most marked differences continued to exist in the various districts. In some, the division of labor was made by parishes in sections of works to be finished within the year. In others, it was divided up into pieces to be carried on during several years so that all should be finished simultaneously. The size of the forces, men, draught and pack animals, was made up by the chiefs of the parishes, and the engineer subdivided the work into tasks or pieces according to the forces. The institution of the "piqueur" and that of the "conducteur"† began to appear in the oversight of the work of those engaged for personal service. According to Gendrier, engineer of the district of Bourges: "the conductor should be possessed of wisdom, honesty and clear-headedness; he should be educated, vigilant, active and endowed with a robust temperament, for this employment requires him to be on horseback at any and all times; to speak, write and work incessantly on the ground." The part of sub-engineer was still more important, as he had charge of the direction of all parts of the work and conductors acted only under his orders. He was constantly on the go, for his department extended over a quarter or a third of a district, and he had to go over the ground many times while work was under way. To all this personnel of overseers was added the "maré-

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*Nothing can be found in dictionary or encyclopædia as to what this punishment was.

†"Conducteurs" of the Ponts et Chaussées follow next after engineers. They correspond nearly to our superintendents. "Piqueurs" are very nearly what we would call overseers.

chaussée"* the presence of which was often permanent on the works. This luxury of inquisitors, reinforced by a not less great luxury of severe penalties is a flagrant proof of the unpopularity of the system and of the frequent mutinies which broke out, among those liable to the service, in spite of all these precautions. There must be added, to these abuses and causes of discontent, the exemptions by which the wealthy classes, the nobility, the clergy, and the well-to-do classes of the cities benefited, so that the toil of the peasants and laborers served to improve the roads which were most enjoyed by those who were exempt. The personal service system must have increased greatly, however, the Service of the Ponts et Chaussées and given to France the great network of roads of which she is proud to-day. Daniel Trudaine, son of a former Prévôt of the Merchants of Paris,† replaced d'Ormesson, in 1734, at the head of the Service of the Ponts et Chaussées, and this indefatigable and intelligent administrator was to organize the Corps of the Ponts et Chaussées very nearly on the same bases as those which underlie it now. As the engineers of the districts did not easily find persons capable of assisting them as sub-engineers and conductors on the works which had become so greatly developed by the personal service system, Trudaine formed in Paris an office of draughtsmen (1744) which he soon transformed into a school for young engineers, giving its direction to Perronet, at that time Intendant of the district of Alençon. So was founded, in 1747, the School of the Ponts et Chaussées, where were kept the maps and plans of the roads of the kingdom. The Corps of the Ponts et Chaussées has perpetuated by means of a bust the faithfully guarded memory of its eminent founder. At the same time, Trudaine laid the first bases of the General Council of the Ponts et Chaussées by forming an Assembly of the inspectors and principal engineers who were at Paris. This assembly met every Sunday at his house. At these weekly reunions the projects for the great works to be constructed, the reports of the inspectors, the projects for competitions by the students at the School of the Ponts et Chaussées, and all the administrative measures which might

*The "maréchaussée" of the time in question were much the same as the "gendarmerie" of the present time in France or the Constabulary in England.

†The Prévôt of the Merchants was a magistrate to whom were referred all questions relating to the commercial life of the city; he had charge also of the fortifications. The position was one of great power and influence and not infrequently dared to hold its own even as against the king.

assist in improving the Corps of the Ponts et Chaussées and the organization of public works were examined and discussed. A decision of the Council of July 7, 1750, reorganized the Corps by raising the number of engineers to twenty-five (one for each district and four at the disposal of the Government for extraordinary or special works), and the number of inspectors to four, these latter having to inspect the entire kingdom, which was divided into four departments. A first engineer and the Director of the School of the Ponts et Chaussées were placed at the head of the service. After the engineers came, as they were needed, sub-inspectors and sub-engineers, and controllers of works, selected from among the students of the School of the Ponts et Chaussées, finally conductors and piqueurs selected from outside of the Corps. The School, under the skilful direction of Perronet, was not long in forming a fine body of young men, thoroughly posted in all the sciences then known as applicable to the art of the engineer. The broad, strong organization of the School, the necessity of showing themselves to be worthy of the positions which they reached successively, the good feeling and emulation among the pupils, the maintenance of the principles of honesty and love of duty gave a high value to the title of engineer, which was won so slowly and with such toil, but which brought with it in high degree the consideration of the public. From this nursery came such names as de Cessart, Lambardie, Fresnel, de Prony, Girard, Collignon, Lalanne, Coriolis, Brémontier, Vicat, Croisette-Desnoyers, Krantz, Jacqmin, de Montricher, Alphand, and so many others which will remain immortal in the history of public works.

At the same time that the Service of the Ponts et Chaussées was reformed and improved, the system of the accounts for public works was also improved and reformed. The royal declaration of December 20, 1762, confirming the principles set forth by the Cour des Comptes,* required that no awards should be made, except on detailed specifications prepared by the engineers and that there should be included in these specifications no matter foreign to the works and no salaries for engineers whose names were taken up on special lists prepared by the Controller General of Finances. Condemnations of contractors were seen also to appear at this

*_Cour des Comptes._ A high administrative tribunal which has charge of checking, verifying, and judging the operations of all who handle public funds; of pointing out to the executive and preparing a statement for the legislative powers, all breaches of laws and regulations.

time, the condemnations being confirmed by the Council of State. For example, the contractors for the road from Metz to Strasbourg were condemned to a restitution of 32,241 livres by the decision of March 17, 1744. The reorganization of the Corps of the Ponts et Chaussées, the adoption of personal service, the energetic impulse from above and the emulation of functionaries caused this period of the old régime to be that of the greatest development of bridges and roads throughout the kingdom. The hard toil and suffering of our fathers have bequeathed to us 6,000 leagues* of broad, straight roads, well built and planted with trees, which since then we have merely had to keep up. The general principles of the *cahier des charges*† as it exists to-day were settled at this time, as is borne witness by the decree of September 7, 1755, in which are summed up every step adopted up to that time. In it are found all the prescriptions relating to condemnation of property, to temporary occupation, to the privileges of public works and to the police of the roads. It was also during this period of thirty-six years (1733-1769) that the construction of the great bridges became greatly extended, at the same time that their strength was made more perfect, as the greater part of them have lasted to the present time. The engineer's art made notable strides forward by reason of the lessons and studies obtained at the School of the Ponts et Chaussées. There may be mentioned, among others, the bridges at Sans, Pont-sur Yonne, Vouvray, Charmes, Port de Piles, Orléans, Moulins, Saumur, Tours, Cravant, Montereau, Mantes, Trilport, Château-Thierry, Saint Edme, Neuilly, built by the greatest names of the period. They were more the works of a school than those of a single man, by reason of the part played by the Assembly of the Ponts et Chaussées which discussed the plans, specifications, and determined the principal lines. The names of Boffrand, Regemorte, Bayeux, Perronet, Hupeau, de Voglie, de Cessart, Pitrou, remain attached to the history of these important

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*One league equals 4 kilometers (2.484 miles).

†*Cahier des charges.* The *cahier des charges* is used particularly by the administration in the case of the award of public works or supplies, grants of railways, sale of any part of the public domain. It shows, for example, in the awards for public works, the guarantees required from the successful bidder, the mode of carrying on the works, the time of completion, etc. It resembles very much our ''instructions to bidders.'' It is placed on view in some public place, where it may be seen and examined by all. The *cahier des charges* differs from one administration to another; the service of the Ponts et Chaussées has one of its own.

works. The annual budget of the Ponts et Chaussées rose also during these thirty-six years to between three and four million francs. This is far from the weakly resources of the beginning, and yet it must be admitted that the work accomplished was below the task demanded from so many arms, so much suffering imposed upon a whole nation by a monstrous despotism. The valuation of this work in days of labor would even have frightened the government which commanded it, so much did it exceed the resources of the budget of Public Works. But this period must not be left without mentioning also the great improvements made in the navigation of several rivers: the Guyenne, the Tarn, the Vire, the Scheldt, the Isle, the Charante, or without recalling the opening of the Neuffossé Canal and the projects for the Burgundy, Berry, Givors, and Ardennes canals and for the lateral canal to the Garonne. Finally, attention must be called to the many works and great sacrifices made to close in the Loire, of which the levee service was reorganized and placed in charge of a chief engineer, as well as to the levees of the Isère and the Drac, the inundations of which desolated Dauphiny. It is from this time that date the first protective works for naval and commercial ports, most of which were added to the Department of the Ponts et Chaussées, especially the works at the ports of Bayonne, La Rochelle and Honfleur.

When Daniel Trudaine died, in January, 1769, and was succeeded by his son, Trudaine de Montigny, at the head of the Service of the Ponts et Chaussées, the personal service, which had been violently attacked by the Marquises of Mirabeau and of Argenson and by several Parliaments, such as the Parliament of Toulouse, had no longer any of the odious characteristics which it had had at the beginning, and the efforts of the intendants and engineers in the several districts tended to mitigate its effects. Two, especially, should be particularly pointed out, in this line, to public attention, Orceau de Fontette, intendant of the district of Caen, and Turgot, intendant of the one of Limoges, proposed, in 1757, to the Assembly of the Ponts et Chaussées to impose works in kind in accordance with the ability of the inhabitants to meet them, but Trudaine objected to the proposition. He then sought to improve the system of tasks by parishes by means of changes which he has set forth in his *Regulations for reducing the burden of personal service.* These changes consisted mainly in a reduction of the tasks in proportion with the increase of distances and in calling upon the communities to chose between work in kind and a pecuniary assistance. These reforms not only assimilated the work in kind

to a tax, but changed also the bases of the work in kind, as the well-to-do taxpayers bore the burden from which the poor ones were released and that which bore on those liable to service in kind but not to the tax. The result was that many complants arose. Orceau de Fontette, for having made the personal service system more equitable and for having done away with its most crying abuses, was prosecuted before the *Cour des Aides** at Rouen. The distress existing at this time in France caused work on the roads to be reduced and led all the provincial courts to denounce everywhere personal service and to demand its suppression. The government was all the time undecided between its desire to have cease the complaints of a much-tried population and that of keeping up the Public Works. The decision of the Parliament of July 19, 1760, however, broke the judgment of the provincial courts, maintained personal service and decided to send troops into the recalcitrant provinces. Orceau de Fontette was able, by firmness mingled with prudence, to hold fast to his reforms, which were appreciated by the people and extended to the districts of Moulins, Rouen, and Alençon.

Turgot, in the Limoges district, tried to have the personal service bought off by a payment of days on the work, but his plan was rejected by Trudaine. Not cast down, he sent to Trudaine, in 1762, a new report developing his whole plan and refuting all the objections of the prime minister. This second report met with no more success than the first. But Turgot, now showing the obstinacy and the tenacity of ideas which characterized his intelligent administration, went on and applied his system to a part of the road from Paris to Bordeaux, by way of Limoges, paying for days of personal service by a corresponding reduction of taxes. Not being able to have his reform authorized by a decision of the Council, he put into operation a series of regulations for carrying out his system, but he was able to set forth his plan of reform with such solid reasoning that he won the acquiescence of the *Court des Aides*. After having multiplied his demands and sent back several times his plan developed at great length and written entirely by his own pen, this persevering man who stopped at no

*The *aide* was an old form of tax levied on articles of consumption, somewhat of the nature of the *octroi* or municipal tax on provisions brought into the city, but of somewhat widér application. The *Cour des Aides* was a tribunal which considered all questions arising in the matter of taxes and not merely of the *aides* alone. It judged also many questions of privileges and exemptions in connection with taxes of all sorts.

work to bring success to ideas which he believed to be just, had the happiness of seeing his reforms indorsed by a decision of the Council of January 11, 1766, which finally covered his responsibility. This decision was renewed annually until the day when personal service was definitely suppressed, in 1787, in the district of Limoges. During these twenty-five years (1762-1787) of the operation of Turgot's system, the works in this district were constantly developing. The cost of the works was more than 300,000 francs; 250 miles of new roads were built and maintained in the best order. The two methods of Orceau de Fontette and of Turgot were not long in occupying all minds. Each one had his partisans and detractors. Both methods, however, were accepted by the populace with enthusiasm, for each was a great step forward in the improvement of that iniquitous tax on labor which was one of the burning questions of this century.

The Corps of the Ponts et Chaussées was extended, under Trudaine de Montigny, in proportion with the development of the public works. An engineer was placed at the head of each of the three departments which formed the district of Paris: Versailles, Fontainbleau, and Compiegne, and fifty commissions* of inspectors were created for all the districts. These inspectors took the place of the sub-inspectors and corresponded to the *ordinaire* engineers of the present day, while the engineers of the districts corresponded to the present chief engineers. The external brilliancy of the Corps was further enhanced, in 1772, by the adoption of a uniform which enabled the public to recognize the various grades of the members of the Corps of the Ponts et Chaussées. The engineers of the Paris district were assimilated, in 1783, to the engineers of the other districts and the number of inspectors general was raised from four to five. The number of inspectors was increased, in 1786, from 50 to 60, and the number of sub-engineers, to which no limit had ever been assigned, passed from 72, in 1771, to 124 in 1784.

When Turgot was appointed Controller General of the Finances in 1774, the first thing to which he gave most earnest attention was to suspend personal service, which had become more and more unpopular and cried down by all political writers and philosophers, and was attacked especially by Voltaire in his *Dictionnaire philosophique*. Turgot's circular of June 6, 1775, stated that this

*Commission must be understood here as of the same nature as the commission issued to an officer of the army or navy.

suspension was only a step toward suppression. This suppression of personal service could only be, after all, a transformation into a new tax, for the resources of the Treasury were not sufficient by themselves to meet the costs of construction and maintenance of the roads and bridges without new funds. But the transformation of the personal service into a pecuniary imposition, applied equally to all, suppressing the iniquitous exemptions and falling on every one according to his wealth, was an immense advance, a real revolution for the period.* Turgot understood this well, for, after endless discussions and consultations with intendants, other ministers and the Presidents of the Parliament of Paris, he could not succeed in finding any one to share his ideas. All the uneasiness and hesitations of the great Minister have been preserved in the fragments of his correspondence with Trudaine de Montigny. Turgot, with his characteristic tenacity, stood by his edict which, in spite of the bitter remonstrances of the Parliament of Paris, was registered by the king sitting in bed of justice at Versailles on May 12, 1776. But the systematic opposition of the Parliament, the remonstrances of the intendants and of the other Ministers and the hostility of the nobility had already nullified it in advance. It was communicated to only four principal Parliaments out of twelve, remained a dead letter, and the great Turgot paid by his fall for the unheard of audacity of wishing to set up in the kingdom an equitable tax, to which every one should contribute in proportion to his means and which placed the lords and the commoners on the same footing.

The first care of Turgot's successor, Clugny de Nuis, was to do away with all his predecessor's liberal reforms and, in spite of the efforts of Trudaine de Montigny, who would have liked to establish a system similar to that of Orceau de Fontette, the royal declaration of August 11, 1776, placed things back where they were before the edict of Turgot. As a matter of fact, the advance of public opinion and the attacks on the personal service as it had been understood originally, did not allow of any strict application of the royal declaration. Personal service was reestablished, but it was greatly improved by a special order of the Controller of the Finances. This order maintained the old privileges and exemptions, but it eased matters up for the poor day laborers by placing

*The road tax (*prestation*), which still exists and of which more will be said further on, was, after all, only a more equitable, more just, more uniformly distributed form of the old personal service in kind.

a maximum of twelve days work a year and brought about, by its various provisions, the suppression of all gratuitous labor.

Trudaine de Montigny was succeeded first by de Cotte (1777-81), then by Chaumont de la Millière (1781-1789). A few additional reforms were made under their direction in the Service of the Ponts et Chaussées. Mention should be made especially of the formation of the first Provisional Assemblies which weakened the authority which the intendants used to have over the engineers of the Ponts et Chaussées, and of the purchase back of the tolls which were not used for the daily maintenance of special works, as prescribed by the decision of the Council dated April 15, 1779. Chaumont de la Millière gave special attention to the School of the Ponts et Chaussées, to which he appointed an assistant director in the person of Inspector General de Chézy, and gave a new set of regulations, of which the essential feature was the obligation of sending one student a year on a mission to Italy. A whole new set of regulations in relation to inspectors, sub-engineers, and students under pay was also drawn up.

Under Necker's ministry, as well as under that of his successor, Joly de Fleury, the question of personal service engaged the attention of the public powers all the time, because it continued to be used very unequally in the kingdom, being the object of unjust vexations in some districts and being almost abandoned or commuted in others. But the discussions of Necker with Perronet, and of Joly de Fleury with Chaumont de la Millière on this subject do not seem to have brought about a firm resolution of reform any more than did the replies of the twenty-four intendants of the kingdom who had been consulted. The struggle between the Parliament of Guyenne and the intendant of the Bordeaux district which occurred during this period of hesitation, which lasted from 1776 to 1786, has become historic. It was for ten years, between a local Court and the Government of the kingdom, a war of decisions published and posted, turn about, by the two parties at issue, without anyone being able to say which of the two would have the last word. Finally, the decree of November 6, 1786, ordained that the conversion of personal service into a pecuniary tax should be tried for three years, and it was followed soon after by the royal declaration of June 27, 1787, which definitely abolished service in kind. Furthermore, events were rushing forward, and the ephemeral and tardy execution of this declaration, which caused one of the sore spots of the old régime to disappear, was not to prevent the French Revolution which broke out two years later.

The difficulties and struggles created during this last period (1769-1789) by the personal service system, did not prevent the advance of public works in many parts of the kingdom. There should be mentioned, however, the better completion and maintenance of the roads by the stone surfacing started by Trésaguet, and through the use of the compressing roller invented by Cassart. New regulations concerning the care and police of the roads were issued; the decree of November 18, 1781, turned over to the cities all the streets and avenues within their jurisdiction, and the decree of April 20, 1783, granted a delay of six months between the laying out of a road and the actual beginning of work, so that adjoining property owners could file and make good their claims for payment for the property condemned.

Among the great navigation works of this period must be mentioned the triple undertaking of the Burgundy, Charolais, and Franche-Comté canals. The Charolais, only, was finished in 1789 The Revolution interrupted work on the other two. The Picardy and Somme canals were also begun, and the projects of the Brittainy and Paris canals were adopted, as was the project for the cut-off of the Yvette at Paris. Must be mentioned, among the works of second order, the rebuilding of the Nevers, Neuilly, and Saint Maxence bridges, the important levee works on the Loire, of which the levee service was simplified by the suppression of the Merchants Company in 1783; finally, the improvement of several seaports, and especially the closing in of the Cherbourg roadstead. The budget for Public Works had strangely increased, also, during this period. From 1769, when it was about five and one-half millions, it advanced progressively to ten millions in 1786 and, if to these ten millions be added the representative value of service in kind, estimated by Chaumont de la Millière at thirteen millions, there is obtained a total amount of twenty-three millions, which is already very respectable.

More time has been given to this sketch of the Service of the Ponts et Chaussées then this division of the work seems to require; but it must not be forgotten that this historical sketch is that of our public works in general, for the other studies to be taken up later, such as the Services of Mines, Railways, Departments and Communes, are of very recent birth and are almost without any history. Hence, this history of the beginnings of our public works was a natural introduction to a work on the present organization of our various Services.

(*To be continued.*)

Japanese Views on the Attack and Defense of Field Fortifications*

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On account of the experiences of the Manchurian War it is natural that great importance should be given by the Japanese army to the matter of the attack and defense of field fortifications. Every year, at one or another of the large maneuver grounds, there is a particular ''pioneer problem'' in which two or three pioneer battalions and also troops of the other branches participate. In this problem a field fortification is built and attacked. The general inspector of military instruction† and the inspector of the pioneers are present at these exercises.

Their criticisms and recommendations are published to all the higher commanders, whose duty it is, in turn, to see that such recommendations are incorporated in the instruction of the troops under their immediate command.

For this purpose exercises are held comprising all three branches of the service, and mostly during February, March, and April, since the young rice plants are planted in May and it is not desirable to occupy the fields then. The weather usually is, at this time, raw and wet. Most of the activities in these exercises occur at night and, as they may last through several days, they are very fatiguing and thus are splendid simulations of war conditions.

The recruits, who in Japan are turned over to their regiments on December 1, can not be used in these exercises. The strongest detachments are formed of old soldiers and consist of about two or three battalions of infantry, one or two cavalry squadrons, two or

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†Since 1898 there has been in Japan a ''General Inspection Bureau of Instruction,'' which is directly under the Emperor and is responsible for the uniform instruction and progressive development of the different arms of the service. The inspectors of the cavalry, field, and foot artillery and of the special troops and also the higher army schools, with the exception of the War College, are a part of this bureau. The inspector general (at present, General Asada) has the right to inspect the troops.

three batteries and two pioneer companies. Supply and sanitary corps are also attached. The Japanese believe that the cooperation of the staff and the line—even behind the front—can only be properly accomplished in war if it has been strenuously practiced in time of peace.

The prescribed principles for the attack and defense of field fortifications are exactly the same for the German and the Japanese armies. In practice, however, several differences arise.

The Japanese regulations prescribe that the defender shall fortify one position only, and that by all available means. In spite of this, there is occasionally a tendency to fortify two parallel lines or else to push out trenches from the main position at various places.

The reliance on long, connected lines of defense has been shattered, as in our (German) army. Only grouped positions are now recognized as practicable. The field fortification manual prescribes, in the main, the building of battalion redoubts or supporting points and field fortifications for a single company. The possibility that these works may be flanked or attacked from various sides is given greatest weight.

The field artillery is also used more than by us in "almost concealed" and "entirely open" positions. By deeper excavations and good masking of the guns it is expected that the early discovery of the artillery will be prevented. The formation of dust clouds during firing is to be prevented by revetting the ground with mats or by means of sprinkling in front of the gun muzzles.

The profiles of the "shelters, covers," and connecting trenches or traverses are of the greatest variety, dependent on the nature of the country, the kind of soil, and the time available for erecting the work. Extremely high value is assigned to building many traverses, which are often made of bags of sand where the soil is difficult to work. Many stepped ramps or exits are cut into the front wall in order to facilitate counter attacks.

The machine guns are assigned the task of supplying the flank fire and are usually installed at the extreme outer angles of the firing line. In every case searchlights are set up beside the machine guns so that the latter may fire on illuminated targets at night.

In addition to tactical requirements arrangements for proper sanitary measures and for the greatest comfort of the troops are also considered. The proper drainage of the trenches is very carefully provided for. As a preventative to moisture, the walls and floors of covered shelters are lined with paulins or rice straw. Charcoal fires, which give but little smoke, are used for warming the men. During the war the men were also provided with hollow shot which had lighted wads inside for warming the hands. Strictest cleanliness is enforced in the temporary field hospitals and in the latrines. During long-continued exercises the ration is considerably increased and provision made for the men to catch up

sleep during the daytime. On account of the close cultivation of every available piece of soil, it is often not possible to make large clearings in the foreground; the more distant positions are, however, marked properly. In addition, masking works and sham positions are laid out, with which the Japanese had marked success in Manchuria. The moving during the night of landmarks which would be of good use to the attacker is considered a fine means of deception.

Particular value is given to the proper location and construction of obstacles. The most efficient obstacle (from actual war experience) is a wire entanglement at least 10 meters wide and not more than 20 or 30 meters in front of the firing line. Its disadvantage is that it betrays the position, unless it is very skillfully laid out and erected. For this reason the greatest of care is taken in masking this obstacle.

For the passage of one's own patrols narrow zigzag paths are left through the entanglement. These paths can be closed against the enemy by *chevaux de frise*. In front of and also behind the wire entanglement land mines are laid. When there is not sufficient time to place wire entanglements, wire loops are placed with which good results were obtained at Port Arthur. The fortification works are carried out on the same lines as our own (German). The pioneers are only employed in the construction of those parts of the works which require trained men; such as "covered shelters" "obstacles," etc. The regulations direct that the work must be pushed vigorously and without cessation from the very first, since experience has shown that the time available for such work is usually underestimated. The important positions are first constructed and then those of secondary importance.

The introductory preparations for the attack on a fortified position bear evidences of the greatest thought. The instructions given in the German maneuver regulations (No. 376 a) that during the daytime infantry detachments should be pushed forward toward the enemy in order to induce him to occupy his fortified positions, and in this way reduce the labor of reconnoitering, are not found in the Japanese regulations. The attacker advances out of range upon the position, conducts a proper reconnaissance and unless an absolute necessity exists to attack by daylight, he delays the further advance of the infantry until dark.

The reconnaissance is done by large patrols composed of pioneer and infantry officers. Its main activities occur during the night, The particular and special duties of the infantry officers' patrols are to reconnoiter the ground of attack with a view to the possibility of a covered advance, the crossing of streams and swampy depressions, and the approximate location of the first attacking position. The pioneer patrols must reconnoiter the hostile position itself. They are ordered to observe particularly from low lying points, in order to see whether or not parts of the (position) works are silhouetted above the sky-line at night; very often the characteristic

silhouette of the wire entanglement betrays the location of field works.

Protected by the presence of the patrols who are working over the ground of attack, the infantry is then brought up in the darkness to the first postion for attack "dependent on the terrain either in closed or open order." With absolute quietness, guided by the fluttering signals of white flags or by signals with dark lanterns the companies advance. A short distance from the selected position for attack a halt is made; the leaders of columns and groups crawl carefully to the front until they believe that they have found points favorable for firing. The laying out or marking of the position is often done by lines made with white strips of paper or by strewing lines with lime. Non-commissioned officers and men to act as guide-marks then remain in the marked position, while the leaders crawl back to bring up the men under cover if possible. The formations used for this purpose differ greatly for different occasions and conditions. On very dark nights, in order to prevent straying of any of the men, the companies are led in line or in marching columns to a wing point of the position, the columns then turn and march along the marked position and take places along its front.

The work of digging the shelter trenches then begins.

While the infantry is pushing forward to this first attacking position, the artillery of the attacker has gone into position farther to the rear.

There seems to be a difference of opinion between the German and the Japanese regarding the necessity of laying out and preparing attacking positions for the infantry, the number of troops and the strength of the field works required. This difference is also noticeable in the printed regulations. Paragraph 380 of the German maneuver or exercise regulations for the infantry states that it is an advantage when the position selected for opening fire is near enough to the hostile position so that the storming of the hostile position can be undertaken direct from it; under such circumstances the first firing position and the storming position will be one and the same.

Based on experience gained in campaign, the Japanese do not figure on the possibility of any affair being so rapidly and quickly terminated. Their regulations state that one should approach a fortified field position by the construction of several attacking positions. In their military literature and in criticisms the thought is expressed that the attacker should consider the field work as "a shield being carried forward," and that in the attack and defense of fortified field works that, on account of the use of shovels and spades, a dissipation of the idea of an attack is to be feared.

So we see that the attacker immediately prosecutes with energy the task of building up the position from which he wished to open the attack by firing. Ordinarily, in strength of profile and in the number of supporting points and bombproofs or overhead cover

these positions are soon but little weaker than those of the defense. The highly developed ability of all arms of the service to construct earthworks brings it about that, during the course of a single night, positions are developed which are supporting points for the further advance of the attack. The development of such strength is, however, ordered also for the defender, who must be imbued with the strongest spirit of offensive defense.

During the advance of the attacker the defense is held in abeyance for the graver operations to follow, and also in order to prevent the early disclosure of his (the defenders') position. Neither does the artillery of the defense open fire as yet. Only some patrols, skillfully hidden in the surrounding country, try to render harmless the first tentative operations of the reconnaissance and information troops of the attacker. The holding back of the defense ceases as soon as the attacker has brought up his troops and begins the work of reconnaissance with larger bodies and in greater detail. Under cover of darkness, and much assisted by their better knowledge of the terrain, numerous strong patrols of the defense fall upon those of the attacker. The searchlights are put into operation, and machine guns and covering troops open fire on the avenues of approach. At places where the attacker advances too rashly or too far, detachments, sometimes large, sometimes small, rush out of the works and surprise the attacking forces. This activity of the defenders compels the attacking forces to push forward strong guarding troops during the preparation of the first firing position and also keeps the working detachments in a constant state of preparedness for battle.

Only in case of very favorable terrain will it be possible for the infantry of the attack to advance still farther on the morning after the building of the first firing position. Ordinarily, this infantry rests at this time, while the attackers' artillery opens fire on the defenders' position. On the evening of the second day the work of advancing is resumed. As the enemy is approached the construction of field works becomes more and more difficult. The use of sand bags is then customary, and it becomes a necessity as one approaches near to the attacking or storming position. Therefore many opportunities are taken to let men rush forward, each loaded with a 20 to 30 kilogram sandbag. From the last firing line to the storming positions a chain of men is formed, at from 3 to 5 paces interval, who lie on their backs, using the cover furnished by every existing small fold of ground. The sandbags are then passed up from the rear and piled by the men farthest to the front. As soon as there is sufficient cover the troops crawl carefully forward and distribute the sacks so that a breastwork is made which acts as cover for the earthwork operation, which is then begun, with naturally a very strong covering force for the working head; and the chain of men is protected by patrols sent out to the front and to the sides. The beams from the searchlights often force the operation to be suspended or else

peculiar means of concealment must be provided, amongst which is the custom of making thick smoke by burning off wet grass, straw, or weeds soaked in petrolium. This method of shelter against searchlights has proven very effective.

The advance into the storming position and the permanent occupation of that position is considered impossible without the use of the attacking artillery. The Japanese regulations therefore prescribe that "before the infantry has occupied the storming position, the artillery must, under cover of darkness, change its position to the front." Ordinarily, at this stage the artillery of the defense is also brought out far enough to the front to shell the attack directly at very close range. Often, as also happened during the war, they take up positions in the infantry line itself.

If, in spite of the fire of the defense and continual sorties, the attacker succeeds in getting firmly into the storming position, he then turns all his energies toward the destruction of the obstacles. As an important principle, it is advised that the strength and manner of construction of these obstacles should be first thoroughly ascertained.

A very careful reconnaissance is therefore made. The total and final destruction of the obstacles is to be made by the pioneers with high explosives. The surest method for attacking the wire entanglement is considered to be that of throwing long, hollowed-out bamboo poles loaded with about 150 sticks of dynamite each, into the wires. These blow out wide streets or openings, which are further broadened during the vital operation of storming. Abatis, and also the wooden part of the wire entanglements are to be destroyed by fire, being first soaked in kerosine. The support and safety of the troops engaged in destructive operations is secured by machine-gun fire in flanking or rear positions. The defense now bends every effort toward the replacing and repair of the destroyed obstacles. Lighting candles, of the nature of roman candles, and straw fires serve for lighting in place of the searchlights, which have doubtless by this time been put out of commission. If there are still searchlights or light projectors on hand they should be installed in the inner ditches or trenches to increase the difficulties of any hostile patrols which may attempt to sneak through weak points in the line. In the last phases of the struggle, when the distance between the combatants has been reduced to 100 meters or even less, the hand grenade comes prominently into use.

In general, the morning twilight is considered (in the German regulations) to be the most favorable time for the execution of a storming movement. This rule, however, is frequently violated and attacks may be made in pitch darkness when it is a question of the capture of weaker outlying positions, or when there is a possibility of penetrating into the defender's lines by quickly following up repulsed attackers.

Behind a heavy veil of patrols, in close order, usually in marching formation, with unloaded rifles and bayonets fixed, the com-

panies advance slowly and silently (about 60 to 70 steps per minute) toward the openings in the obstacles. When the first advance troops have reached the obstacles, they throw themselves on the ground. Under cover of their fire, pioneer troops with wire cutters and small explosive cartridges try to widen the openings. Through these openings the columns rush forward to the attack. Deploying toward each side when past the obstacle they rush at the enemy in close order, silently, without cheering or outcry. The defender does not await the attack in his trenches. If the attacker has succeeded, in spite of the heavy fire of artillery, machine guns, and rifles, in spite of the explosion of hand grenades and land mines, in penetrating beyond the obstacles, then the defenders rush out to meet the attack with the bayonet. The decisive struggle occurs in the narrow space of 20 to 30 meters which separates the trenches from the obstacles.

If the attacker is victorious he is satisfied to occupy the defender's position. He quickly alters the works to make them front in the opposite direction and to secure himself against counter-attacks. The Japanese never neglects to prepare for counter-attacks when he has any reserves at all left. He knows that this is the most propitious moment for such attacks. The attacker has suffered heavy losses, his communications are in great disorder; there will also be a tendency to relax in watchfulness and vigor of operation after several days and nights following the heaviest labor.

The attacker therefore draws his reserves, who have in the meantime occupied the storming position, into the captured position. Artillery and machine guns hurry there also. The communications are put in order, but only in full daylight is the pursuit taken up. Frequently the attacker does not succeed in his first attempt at capturing the defender's position. With heavy losses and in great disorder he falls back, followed hotly by the counter-attacking defenders. In this case, his attacking positions prove of inestimable value to him. With their help he will succeed in making a stand and preparing for a fresh attack. Thus they actually act as "A shield, which the attacker carries before him."

Editorial Notes

The Memoirs in 1912

One year ago we announced that the number of paid subscribers to the PROFESSIONAL MEMOIRS was four hundred, an increase of 11 per cent in two and one-half months following the inauguration of a small campaign of publicity by sending out circulars to selected individuals whom it was believed would be interested in the subjects covered by the PROFESSIONAL MEMOIRS. This campaign of publicity has been kept up throughout the year with very satisfactory results, inasmuch as the net paid circulation has increased over 50 per cent during the year and bids fair to continue at the same rate.

This, of course, has resulted in an increased income and, as no profit is made on the publication, it will enable more money to be put into cuts and pictures than heretofore, this being one of the most expensive parts of getting out the publication. We have other plans for improving a little, from time to time, the PROFESSIONAL MEMOIRS, and as time and money become available they will be put into effect.

We desire at this time to express our appreciation of the support given to the magazine in the past, and trust that its excellence will warrant the same support in the future. We would state for the benefit of our readers at large that we will be very glad indeed to receive articles from anyone, and we desire again to call attention to the offer of prizes made in No. 18 for the year 1913; that is, one hundred dollars for four prizes of fifty, twenty-five, fifteen and ten dollars each for the four best articles published during the year, the competition being open to all subscribers excepting officers of the Corps of Engineers with more than ten years' service in the Corps. It is hoped that the award of the four prizes for 1912 can be announced in the next issue. We will gladly send information, upon request, as to how articles should be prepared, and if each one intending to write an article will make such request it will save us and possibly the author himself considerable work and

delay. This is particularly important as regards maps and drawings.

Book Reviews

We have received three or four seemingly very good books for review, but due to lack of time to review them properly for this issue, we are obliged to carry them over to the next number.

Another Lock Accident in Welland Canal[*]

A serious accident and break in the Welland Canal occurred about two o'clock on the afternoon of November 1, when the steamer *Samuel Marshall*, owned by the Central Canada Coal Co. of Brockville, carried away the four gates of lock 13, and badly damaged the bridge crossing the canal a short distance below the lock. The *Marshall* was bound up, light, and on entering the lock rammed the two head gates, throwing them apart and letting the waters of the upper level rush through; tearing the upper and lower gates from their sockets and forcing the steamer back against the bridge which crosses the canal. The working gear of the bridge was badly wrecked, and the bridge thrown out of position, blocking all travel along the road. Fortunately, the level above is a short one, and there was comparatively little damage to the canal banks or surrounding country. The steamer escaped without serious damage. The ponton, with new gates, from Port Dalhousie, was at once hurried up to the break.

There has been an unusual number of accidents this season on the canal, this being the fifth time that gates have been carried away.

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*From *Marine Engineering of Canada*, for November, 1912.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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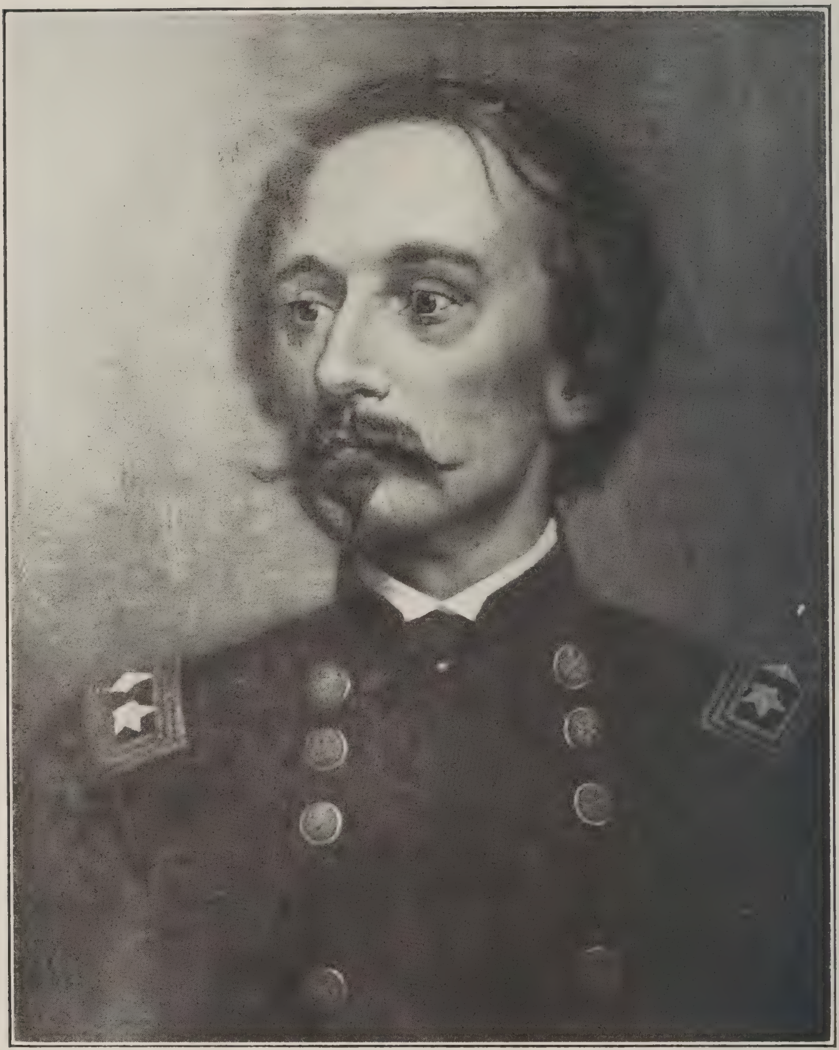


BREVET MAJ. GEN. GOUVERNEUR KEMBLE WARREN

CORPS OF ENGINEERS, UNITED STATES ARMY

1850-1882

BORN 1830—DIED 1882

SEE P. 199

The Georgetown Reservoir

BY

Lieut. J. J. BAIN
Corps of Engineers

The Georgetown reservoir is located in the District of Columbia, at the intersection of the Conduit Road with Reservoir Street.

In shape it is almost a rectangle, approximately 2,250 feet long and 850 feet wide, including some 42 acres.

It was constructed as the distributing reservoir for the system of water supply for the District of Columbia, designed by Capt. M. C. Meigs, Corps of Engineers. It was Captain Meig's intention to divide the reservoir into two sections, the upper containing about 24 acres and the lower about 18 acres. The division was to have been made by an embankment, the top of which was to be at an elevation 2 feet below the ordinary surface of the water in the reservoir. By this design, the upper section would have become a sedimentation basin, and the lower section a storage basin where the pure surface water flowing in a thin stream over the top of the dividing embankment would be stored until drawn off from the opposite end of the lower section for consumption in the city of Washington. Actual construction was begun under contract in September, 1862, but in one regard a radical departure was taken, by those then in charge, from the plans of Captain Meigs. The dividing bank of the reservoir was built to the full height of the outside banks, a narrow cut lined with masonry and fitted with stop plank openings was left in the middle of its length, and a 48-inch cast-iron pipe line was laid from this opening beneath the bottom of the lower section to the effluent gatehouse. By this arrangement water could be drawn from the lower section after having passed through both sections, or it could be drawn direct from the upper section through the cast-iron pipe without passing through the lower section.

For many years the reservoir served its double purpose as a storage and settling basin, but as the consumption of water in-

creased, it was noted that its sedimentation duties were not performed as they should have been.

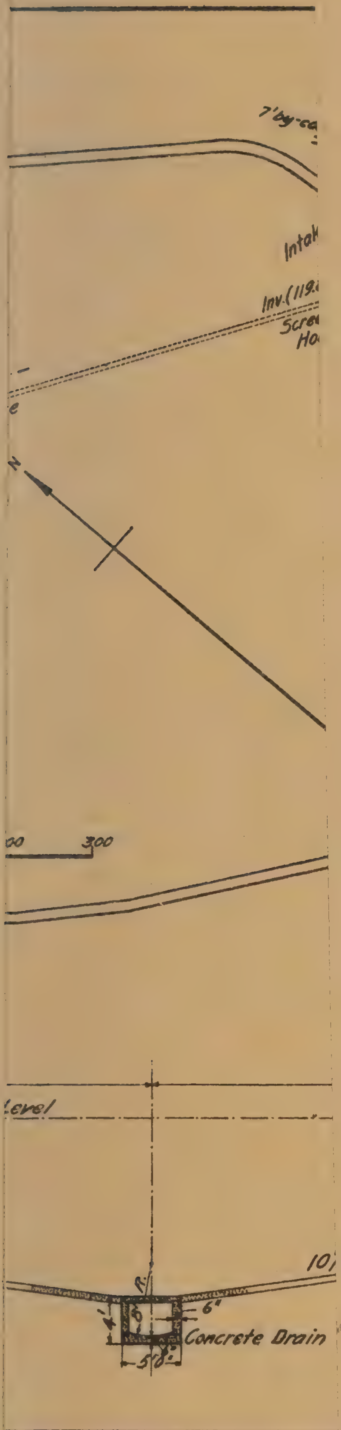
The draft through the middle opening of the dividing embankment and the cast-iron pipe and through the effluent gate was so great that the greater part of the water which reached the city mains was drawn in a comparatively narrow stream straight from the influent to the middle gate, and thence straight to the effluent gate. In times of turbidity, the muddy water followed this stream and did not diffuse itself through the whole body of water in the reservoir, as it should have done in order to produce the greatest amount of settling. It was at one time recommended that the reservoir be altered so as to conform to the original design of Captain Meigs, but this was not done.

When the filtration plant was put in operation on August 21, 1905, the Georgetown reservoir ceased to be the distributing reservoir of the system, but was continued in use as an intermediate storage basin. It was found that while the filtering reduced the turbidity to a great extent, it did not entirely remove it at all times, as some of the silt in the Potomac water is very fine and is not completely intercepted by the sand of the filter beds when the water is extremely turbid.

In 1910, Congress authorized the construction of a plant for the application of a coagulant at certain times in order to reduce the turbidity of the Washington water. This plant was constructed on the line of the conduit, just below the Dalecarlia reservoir. This made the remodeling of the Georgetown reservoir absolutely necessary, since, according to the theory of its action, conditions favorable to thorough sedimentation must exist in order to get the full benefit of the coagulant.

In the annual appropriation bills for 1912 and 1913, Congress appropriated $108,000 for the purpose, and the work of remodeling is now being done.

The approved project for the expenditure of the above appropriations requires, in general, the following changes to be made in the reservoir: *first*, stopping up the old opening or cut in the middle of the cross dam; *second*, cutting a new opening near the extreme southern or right-hand end of this cross dam; *third*, the erection of an earthen dam as shown in Plate I, starting from the upper end of the reservoir near the influent gate and running in a general way parallel to the left-hand or Conduit Road side of the reservoir until it intersects the cross dam; *fourth*, the construction



Intak
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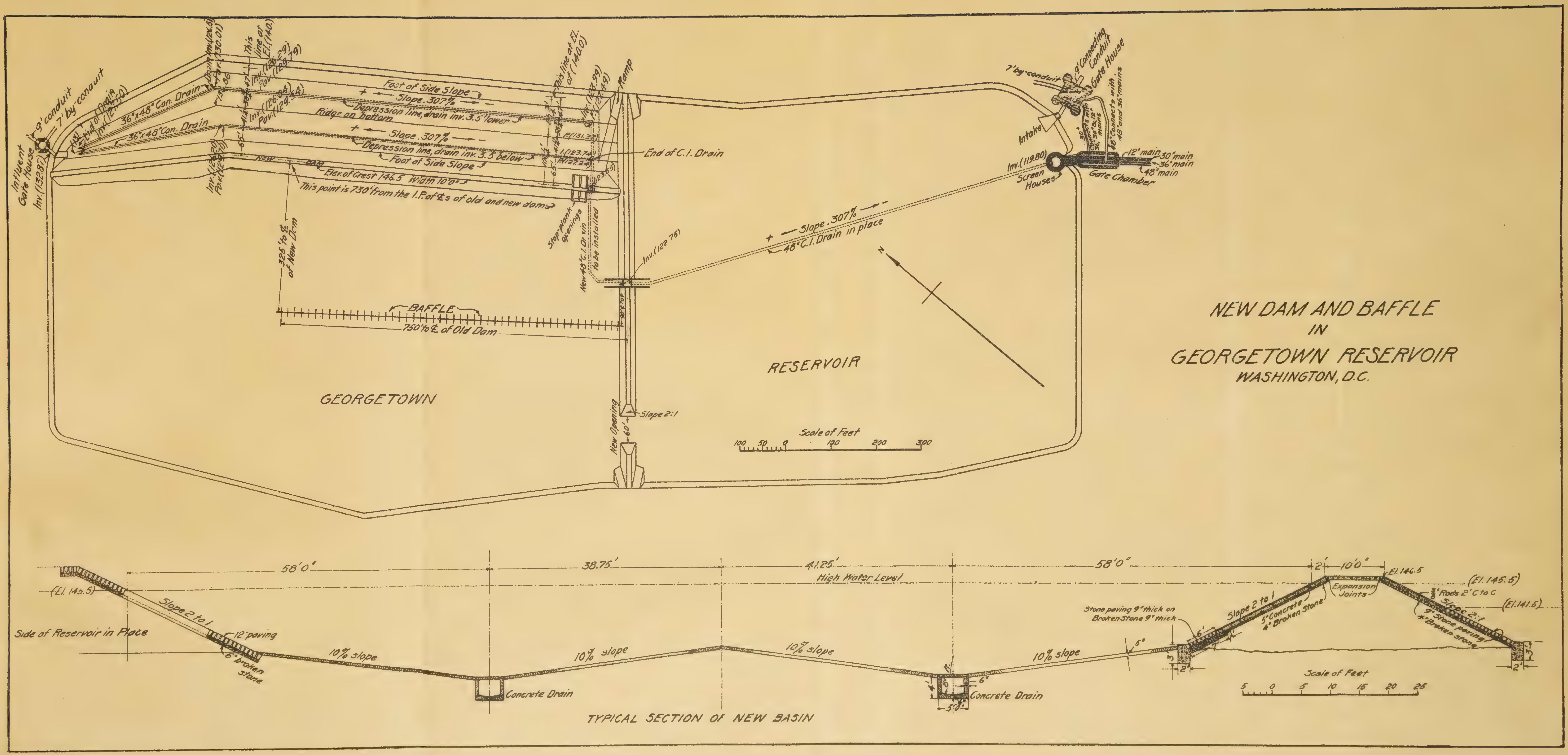
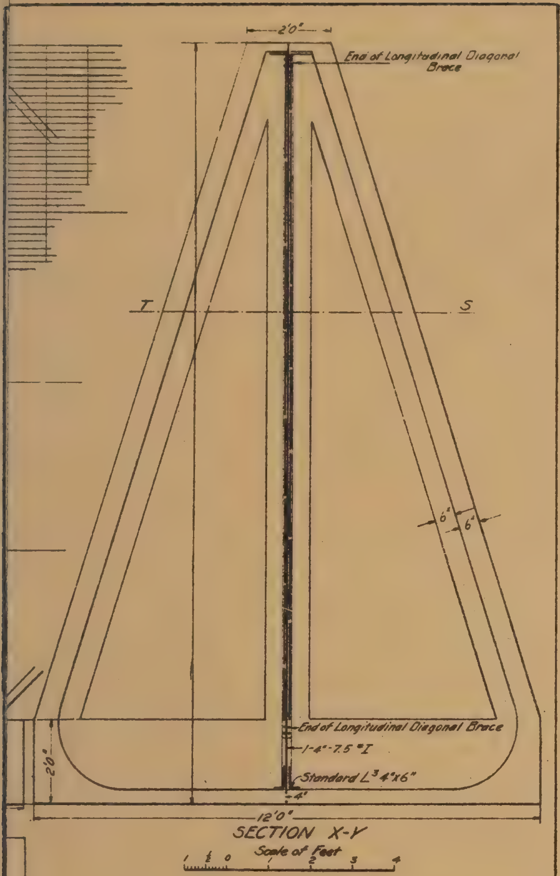
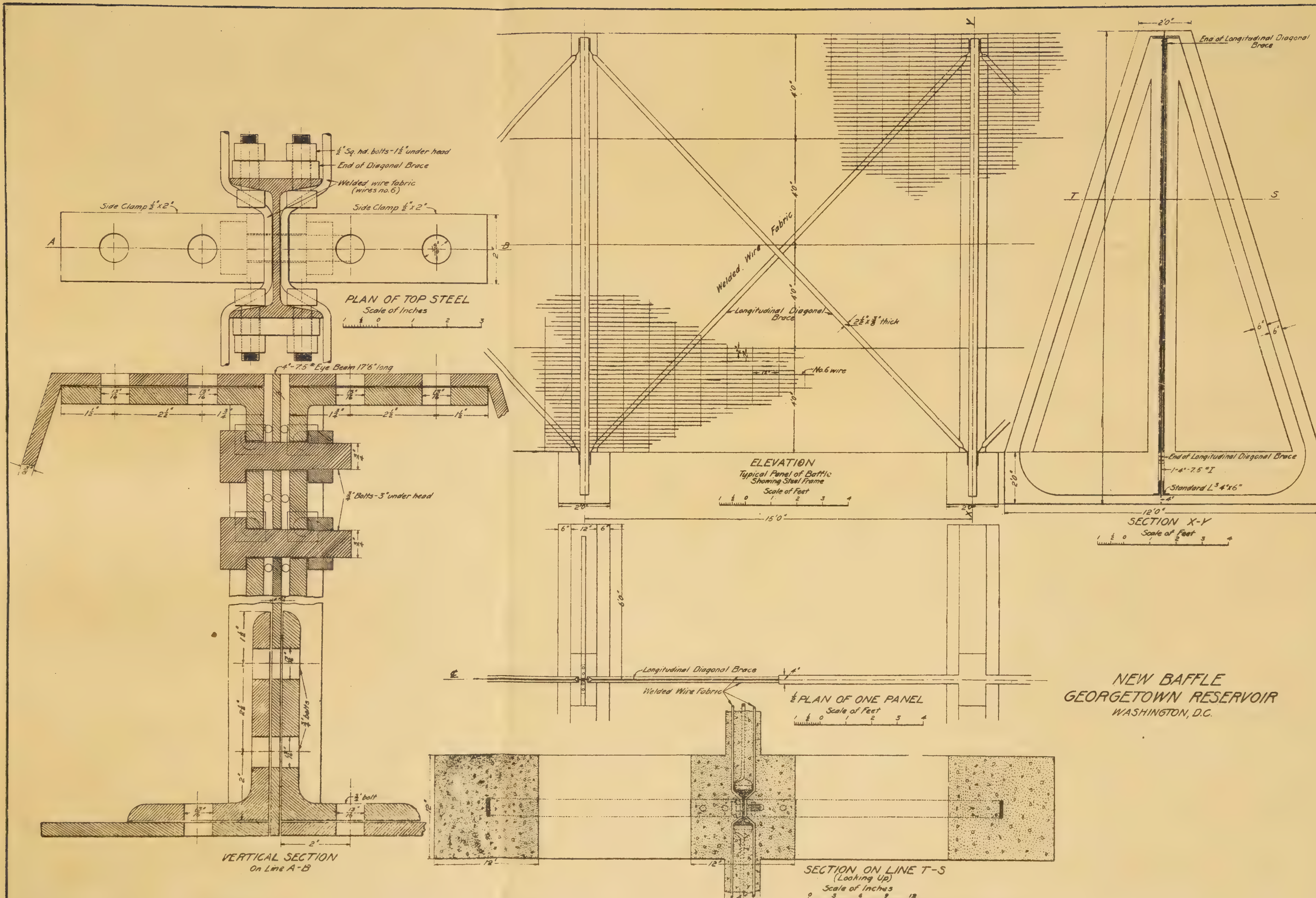


NEW DAM AND BAFFLE
IN
GEORGETOWN RESERVOIR
WASHINGTON, D.C.
GEORGETOWN
RESERVOIR
BAFFLE
750' to ℄ of Old Dam
326' to ℄ of New Dam
Elev. of Crest 146.5 Width 10'0"
This point is 730' from the I.P. of ℄s of old and new dams
Foot of Side Slope
Slope .307%
Depression line, drain inv. 3.5' lower
Ridge on bottom
36"x48" Con. Drain
Depression line, drain inv. 3.5' below
End of C.I. Drain
Slope .307%
48" C.I. Drain in place
New 48" C.I. Dr. to be installed
Stop-plank openings
New Opening
Slope 2:1
Ramp
Influent Gate House
9' conduit
7' by-conduit
7' by-conduit
9' Connecting Conduit
Gate House
Intake
Screen Houses
Gate Chamber
12" main
30" main
36" main
48" main
Scale of Feet
100 50 0 100 200 300
58'0"
38.75'
41.25'
58'0"
10'0"
High Water Level
(El. 145.5)
El. 146.5
(El. 145.5)
(El. 141.5)
Side of Reservoir in Place
Slope 2 to 1
12" paving
6" broken stone
10% slope
10% slope
10% slope
10% slope
Concrete Drain
Concrete Drain
Stone paving 9" thick on Broken Stone 9" thick
Slope 2 to 1
5" Concrete
4" Broken Stone
Expansion Joints
Slope 2:1
9" Stone paving
4" Broken stone
Scale of Feet
5 0 5 10 15 20 25
TYPICAL SECTION OF NEW BASIN



NEW BAFFLE
GEORGETOWN RESERVOIR
WASHINGTON, D.C.



NEW BAFFLE
GEORGETOWN RESERVOIR
WASHINGTON, D.C.
PLAN OF TOP STEEL
Scale of Inches
VERTICAL SECTION
On Line A-B
ELEVATION
Typical Panel of Baffle Showing Steel Frame
Scale of Feet
SECTION X-Y
Scale of Feet
½ PLAN OF ONE PANEL
Scale of Feet
SECTION ON LINE T-S
(Looking Up)
Scale of Inches
Side Clamp ½"x2"
End of Diagonal Brace
Welded wire fabric (wires no. 6)
4"-7.5# Eye Beam 17'6" long
¾" Bolts-3" under head
Welded Wire Fabric
Longitudinal Diagonal Brace
No. 6 wire
End of Longitudinal Diagonal Brace
Standard L³ 4"x6"
15'0"
12'0"

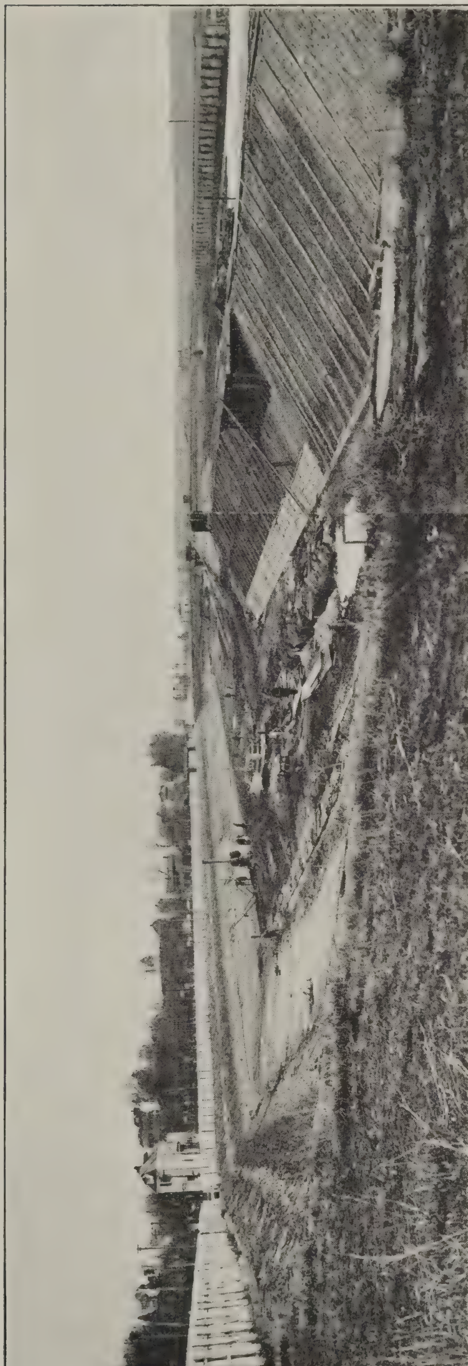


Fig. 1. General view from influent gate; settling basin directly in front; new earth dam at right center and baffle at extreme left; effluent gatehouse from reservoir is round building near center of sky line.

Fig. 2. General view from north bank of reservoir near the north west corner, showing baffle, new earthen dam with sluiceways near south or far end, visible about three-fourths inch to left of free end of baffle.

Fig. 3. Baffle, from north or free end.

of a gap or cut in the lower end of the new dam, containing eight sluice-gate openings; *fifth*, paving the top and sides of the earthen dam with concrete or riprap; *sixth*, the construction of two longitudinal concrete drains from the inlet to the outlet of the sedimentation basin; *seventh*, paving of the sedimentation basin with concrete; *eighth*, the construction of a reinforced concrete baffle parallel to the new dam in the portion of the upper section outside of the new sedimentation basin, and, *ninth*, laying a new section of 48-inch cast-iron drain to connect that already laid with the lower ends of the concrete drain in the sedimentation basin.

The work was begun in March, 1912, and will be completed, partly by contract and partly by hired labor, by June 30, 1913.

The most novel feature of the construction is the baffle, the details of which are shown in plate II. It consists of open reinforced concrete triangular piers, spaced 15 feet apart, and connected by a thin concrete wall, or web.

The piers were made by placing the steel reinforcement and concrete in forms in the usual way. The web between piers was built up by shooting mortar, by means of a cement gun, against wooden forming placed behind the outer layer of wire fabric reinforcement until the web had been built up to a thickness of about 4 inches.

After making the above-named changes, the coagulated water will enter the sedimentation basin through the influent gate, and will remain there long enough for the coagulating action to become complete, thus depositing the greater part of the coagulated matter upon the concrete floor of the basin. By means of the sluice gates the water may be drawn into that part of the north half of the reservoir outside the sedimentation basin, either from near the surface or from near the bottom. Before reaching the effluent gate the water so drawn must travel in a tortuous path around the ends of the new baffle and the end of the old cross dam which originally divided the reservoir into two parts. It is believed that by this arrangement the water will be given more time to complete sedimentation, and will diffuse itself more thoroughly over the reservoir, because, while its velocity is the same as before, it travels over a longer path.

For cleaning purposes, the sluice gates between the sedimentation basin and the remainder of the north basin will be closed and the one in the 48-inch drain opened. Such of the sediment which is not drawn out with the water then in the reservoir, will be

loosened and carried out by flushing with fire hose. By a new connection at the effluent gatehouse, the sediment bearing water will be conducted back to the Potomac River by way of Foundry Branch.

Fig. 4 (top). Effluent gatehouse. The building with the dome was General Meig's design; the building in the form of a castle was built later.

Fig. 5 (bottom). West side of new earthen dam, showing sluiceways between the settling basin and remainder of the north half of the reservoir. Influent gatehouse with dome can be seen near left edge of illustration.

Fig. 6 (top). Right, or east side, of new earthen dam, showing sluiceways, concrete and stone paving and part of settling basin with openings into one of the drains for flushing the basin.

Fig. 7 (bottom). East, or right, wall of the settling basin, showing original paving at top and one of the flushing drains at the bottom.

Improvement of Rivers*

BY

Maj. WM. W. HARTS
Corps of Engineers; Member American Society Civil Engineers

In undertaking any river improvement for the benefit of navigation, many considerations necessarily enter into the determination of the methods to be followed. It sometimes occurs that the solution is so obvious as to be self-determining, as at the Cascades of the Columbia River where the advantages of a short lateral canal across a convex bank were so plain as to practically exclude other methods from consideration; but in the usual case there is a choice among several methods and usually considerable study is required to select the most suitable. This selection often depends more on the character of the river than it does on the nature of use to which the work will be put.

For example, at Rock Island Rapids in the Mississippi River, where the channel has been successfully used for many years as the result of open river regulation, it was at first thought that a lateral canal would be necessary and such an improvement was proposed, even for the depth of 4½ feet then needed. But a fuller study of the problem showed that the character of the river was such that the needs of navigation, as then existing, could be adequately met in an artificial open waterway supplemented by dikes and other contraction works.

The influences affecting the choice of a method of river improvement are changing, being constantly modified by later experience and by newer and better mechanical appliances. From present tendencies there seems small doubt but that in many instances lateral canals would probably no longer be so freely chosen at the present day as formerly, but that some type of canalization would be adopted instead if the project were up for determination anew.

The reasons which may incline the engineer to the selection of any particular type of improvment may be analyzed with advantage. The marked advance in recent years in the efficiency of machinery for excavation is one of these reasons which has more or less changed the economy of channel building and has had its

*Reprinted, by permission, from Proceedings of XIIth International Congress of Navigation, held in Philadelphia, May, 1912.

effect on the choice of methods to be followed. Many harbor channels that originally could not be deepened, owing to the high cost of effective improvement, are now being dredged to depths suitable for deeper draft vessels. In New York Harbor the Ambrose Channel, 7 miles long, 2,000 feet wide and 40 feet deep, is being dredged with sea-going hydraulic suction machines at the surprisingly low total cost of 5.4 to 5.7 cents per cubic yard (Professional Memoirs, Engineer Bureau, U. S. Army, January-March, 1909, pp. 61-62.) Some years ago it was believed by many engineers that an effective entrance channel to this harbor could only be secured permanently by the adoption of protecting jetties of riprap. The high cost of such work and its physical difficulties deterred engineers from undertaking it for a long time, and only within recent years has the construction of the entrance channel been considered economically possible, mainly through the greater perfection of dredging machinery.

This same tendency is felt to a certain degree on our rivers, and our ideas of improvement are likewise undergoing some readjustment both as to practicability and as to choice of methods. Although this choice is often not as free as might be liked, since local conditions sometimes place narrow limits upon it, still, whenever one of several plans is to be selected for adoption the changing circumstances must be borne in mind.

For example, soft digging with a dipper dredge was done on the Lower Tennessee River in 1910 for three and six-tenths cents per yard, place measure, including all current field expenses, but excluding extensive repairs, plant cost and overhead charges. At Muscle Shoals Canal a record of nineteen years shows that sediment has been removed from the canal at the same average cost with a ladder or continuous bucket dredge. Including all charges, the cost has been 5.16 cents per yard at the latter place. In excavating through rock ledges exceptionally reasonable work has been done at Allens Bar near Hobbs Island, where the entire cost, including blasting, dredging, loading on barges and dumping in dikes amounted to only 28.1 cents per yard, including all current field expenses. These low prices are undoubtedly largely due to careful management, but improved machinery is nevertheless the important factor.

Before discussing the methods of river improvement, it may be desirable to enumerate the various classes of work used. The four principal divisions of river improvement methods are as follows:

1. Contraction, including the use of spurs, sills, training walls and bank protection.
2. Excavation, including dredging.
3. Canalization, including locks and dams; and
4. Lateral canals.

These methods are all well recognized and are in extensive practical use to-day. To these is sometimes added a fifth: Reservoirs.

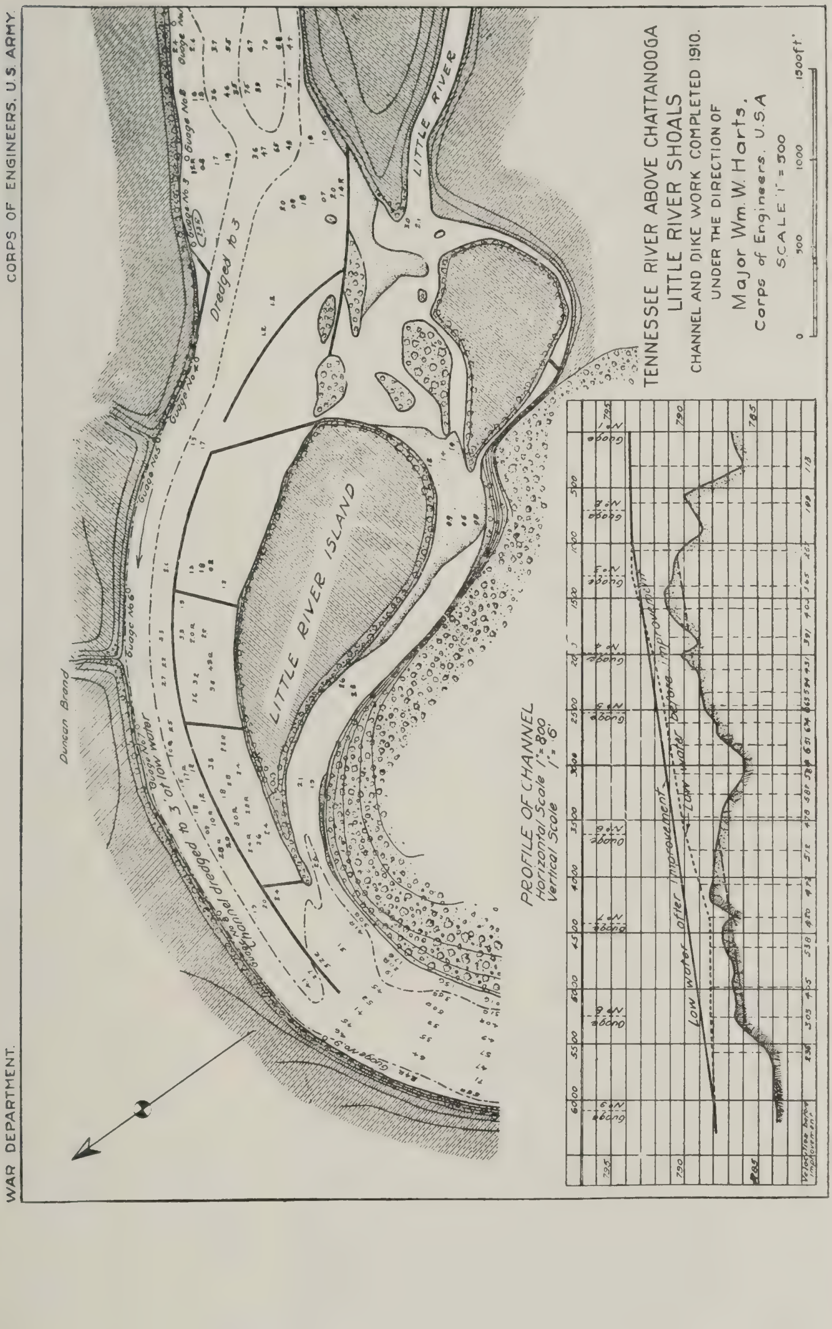

WAR DEPARTMENT.
CORPS OF ENGINEERS. U.S. ARMY.
TENNESSEE RIVER ABOVE CHATTANOOGA
LITTLE RIVER SHOALS
CHANNEL AND DIKE WORK COMPLETED 1910.
UNDER THE DIRECTION OF
Major Wm. W. Harts,
Corps of Engineers. U.S.A
SCALE 1' = 500
0 500 1000 1500ft
LITTLE RIVER ISLAND
LITTLE RIVER
Duncan Brand
Dredged to 3
Channel dredged to 3' at low water
PROFILE OF CHANNEL
Horizontal Scale 1'=800
Vertical Scale 1'=6'
Low water after improvement
Low water before improvement

Fig. 1.

Engineers are ordinarily largely guided by their preferences for certain classes of work, usually those met with in their own experience, and are often inclined to look with some disfavor on methods with which they are less familiar. But it would undoubtedly be best to recognize at once the good points of each tried method and combinations of two or more of them whenever found by experience to be advantageous. Untried theories and purely experimental modes of improvement will usually not receive extensive application at the hands of practical men.

On one point, however, most engineers will doubtless be agreed, that is, that the navigable part of the river must be studied as a unit rather than piecemeal, and work must be done with some definite coordination, so that a logical, systematic, and connected improvement will result.

No two rivers are just alike in slope, discharge, character of bed and banks, nature and size of drainage basin, or kind and amount of sediment carried, but, in general, it is observed that they are alike in some particulars, that they all have a more or less winding course with a constantly varying hydraulic radius, and consist of a succession of pools where depths are greater than the average, separated by bars where depths are less than the average. The fact is now no longer overlooked that these bars are like submerged weirs and have a decided effect on the elevation of the water surface. The danger of the early methods of improving the Rhône by deepening shoals separately, and cutting through bars independently of their relationship to the remainder of the river, has been repeatedly dwelt on by engineers. Unless contracted artificially, this invariably facilitates the outflow of the water of the pool above the site of the work, with a consequent lowering of the surface, and often causes an ultimate decrease instead of an increase of depths. On the Rhône at La Mulatière the level of the low water surface was reduced 4.66 feet in seventeen years by a deepening of the channels on the shoals below.

It seems plain that each separate work in a river should bear a similar relationship to the others as do the links of a chain. This need not prevent the gradual application of the methods selected, but would require that each work should form a part of a definite, comprehensive scheme.

The use of contraction works seems more particularly adapted to the portions of rivers where ample discharge, considerable width, small slope, and gentle flow are met with. The Rhine below Strassburg, the Elbe from its entrance into Germany, the Niemen through Prussia, the Rhône below Lyons, are foreign examples of rivers that have been improved by these means. In this country the Mississippi above the mouth of the Missouri is an excellent example of this class of work, and instances of its successful application are found in the Columbia, Missouri, and Tennessee rivers, and on the French Broad and Hiwassee, tributaries of the Tennessee.

Fig. 2. View of contraction works at Little River Shoals, Tennessee River, looking upstream from below end of dike.

The amount of depth attainable by this method is ordinarily very limited. If the contraction is overdone, in rivers with movable beds, an excess of scour may result, accompanied by a lowering of the water surface and perhaps a flattening of the slope in the lower reaches; or, if the beds are formed of resisting material, an excess of velocity may be occasioned.

It is an elementary principle that movable river beds must ordinarily be protected wherever contraction works are used, for otherwise the effect is sure to be largely local and the material scoured out by the currents is likely to be deposited elsewhere, frequently on other bars where depths were ample before. The protection of the banks in such cases is usually far from simple, but the bottom protection is often much more difficult and costly. Sometimes this arises from the lack of suitable materials, sometimes from the width of river, sometimes from the character of the material of the bed and banks, but oftenest from the soft and shifting nature of the bottom. For these reasons, the regulation of the Ohio by spurs and training works has been abandoned. Until sills for the protection of the river bed were adopted on the Rhône in 1882 the full benefit of the many years of regulation was not obtained.

It is doubtful whether more than a very few feet can be secured by this means in the average case without overdoing the amount of contraction. It has been stated by Prof. H. Engels, a well-known writer on this subject, that: "1. Only rivers or long reaches of rivers in which natural erosion is fully developed are adapted to regulation. The navigability of unfinished rivers yet in a state of erosion can be improved with permanent results only by canalization.

"2. The *most* that can be accomplished by regulation is the *desired adjustment* of the slope of the low water line, and this only on reaches of uniform regimen and uniform characteristics.

"3. This feasible adjustment of the slope to be accomplished when the conditions are most favorable can only be established and brought about by constructive measures after the formation of that part of the channel which rises above low water is completed; after the conditions of the bed have adapted themselves to the change of energy caused by the formation of the mean and high water bed; in other words, after the erosion caused by this formation has come to rest.

"4. To secure the establishment and permanent preservation of the adjustment of slope, the irregularities of the bed in the longitudinal and transverse profiles are to be adjusted after reinforcing the low water shore, and the bed is to be strengthened where attacked by the water on account of the ground plan of the channel. Restriction of width alone will not bring about that degree of navigability which may be attained."

He also states that the depth attainable is expressed in the for-

mula $d = \left(\frac{Q}{wk\sqrt{i}}\right)^{\frac{3}{2}}$ in which d is the depth; Q, the measured discharge; w, normal low water width; i, the adjusted slope, and k a constant corresponding to the constant of Chezy's formula. (Trans. Am. Soc. C. E., Vol. XXIX, page 220.)

If these views seem rather extreme as far as they bear on "unfinished" streams, the assumption that the gain is measured by the practicable amount of adjustment of the low water slope is open to much less question.

Dredging as an exclusive method of original improvement is seldom practiced on interior rivers of the United States, except where the banks are comparatively low and unstable, the flow very gentle, and the discharge large. Navigable depths of 9 feet in the Mississippi River below the mouth of the Ohio are now being maintained mainly by dredging, but the work is being supplemented by bank protection, the design being ultimately to protect the banks from caving by the use of brush mats and riprap stone, and thus finally limit the bank-full channels to a predetermined width in order to accelerate the carrying velocity and insure regularity of regimen. This will also aid in maintaining depths and, perhaps, finally render much of the dredging unnecessary. Ten suction dredges of great capacity are employed in this work. It has been estimated from the surveys of 1908 that there were 749 miles of caving bank along the length of 790 miles of river between Cairo and the Red River, almost equivalent to one entire bank, and observations in 1892 indicated that 100,000 cubic yards per mile fell into the river annually between Cairo and Donaldsville, La. It seems plain that until the banks can be protected nothing permanent or satisfactory can be expected. A recent plan for increasing this 9-foot depth to 14 feet proposes regulation of the low water slope by the addition of transverse sills combined with a contraction of the banks. Above the mouth of the Ohio and up to the mouth of the Missouri 8-foot depths at low water are maintained by dredging supplemented by the use of permeable dikes to restrict the river width at the bank-full stage to 2,500 feet. These permeable dikes encourage the deposit of silt and the growth of new banks.

Open river regulation is usually the first method studied for new projects. If the depths required for boats are not too great, if the river is wide and dams for canalization expensive, if the banks are low requiring too many low-lift locks, and especially if funds are not sufficient for any other more radical improvement, resort is usually had to this kind of work. After a general scheme has been adopted work can be carried on at the difficult places first and in this way the most urgent needs of navigation met in order of importance.

Contraction works leave the greater part of the river open for boats, thus avoiding the delays at locks, are generally moderate in cost, can be changed if not entirely successful upon the first trial,

may be applied gradually, do not flood the banks nor interfere with high water flow, and ordinarily indicate the channel to pilots at low water. On the other hand, they require careful study for correct location, are not invariably successful, require considerable maintenance, and are usually limited in effect to a very few feet. They are not adapted to rivers of small discharge and steep slope.

Following European practice, the efforts to obtain 6-foot navigation on the Ohio River above Cincinnati were directed during the early 80's toward improvement by dikes and training walls, of which many were built at the various shoals. In plan they left the bank in a long curve and followed downstream parallel to the channel, contracting the waterway without protecting the river bed. They were built to a height of 4 feet above low water and their completion caused a disturbance in the equilibrium theretofore existing and was usually followed by a local increase of depths, sometimes at the expense of decreased depths on bars lower downstream where the dislodged material was occasionally brought to rest. This method was only partly successful in the upper portions where the low water discharge was small and shoals frequent. When the 6-foot project was abandoned and the 9-foot project adopted for this river, the more radical method of locks and movable dams was selected and the use of training walls abandoned as insufficient.

Dredging in rivers, on the other hand, is immediate in its effect, the expenditure is applied directly to the seat of the trouble, and it is suitable for any type of stream. Dredged channels are usually not difficult to plan and their excavation is seldom accompanied by special engineering difficulties. When well located they often are fairly permanent. If the shoal is caused by a deposit of sediment it will sometimes reform, unless prevented by contraction works so designed as to remove the cause of the original deposit. On the Mississippi redredging each year is still necessary on some shoals, but in hard river beds elsewhere the channels are often reasonably permanent. It is frequently the practice to use the excavated material in constructing dikes or training walls for a compensating contraction of the river in the neighborhood of the dredged cuts wherever the material is suitable, as is done on the lower Tennessee with good results.

If either dredging or contraction should be followed in river improvement to the exclusion of the other a considerable advantage would be lost, for it is by the combination of these two methods that the best open channels have been obtained in river work.

Regulation by spurs, sills and training walls, supplemented by dredging, merges without any sharp dividing line into dredging supplemented by contraction works, and the predominance of one method over the other will depend invariably on the character of the stream and the results to be achieved.

As a type of work where the contraction feature is the more predominant, the Upper Tennessee is a fair example. There the

Fig. 3. General view of Lock and Dam No. 28, Ohio River, during construction, December 29, 1912. Lock walls and gate recesses nearly completed.

present project is to obtain a 3-foot depth at low water by open channel work. The methods followed to-day represent the outcome of many years' experience with dikes, bank protection, spurs and channel excavation and will be briefly described. The river has a normal width of about 600 feet, discharge of about 2,500 second-feet at low water, and 395,000 second feet at extreme high water. The bottom on nearly all shoals is rock or hard gravel overlying rock at varying depths. The variation in bed and banks is so small from year to year that it may be justly described as of "fixed regimen."

The preliminary step is always a careful, detailed survey, showing the depths, nature of the bottom, low-water slope, and velocities at various places. A study of the physics of the locality is made and the action of the currents at varying stages observed.

The main steps in the work are then as follows:

First, the secondary channels back of islands are usually closed and the best location for the navigation channel selected. This location is almost invariably along one bank, the reasons for which are numerous, the following being the principal ones: It is more easily navigated, especially at night, more easily found by pilots, and more easily maintained, as it usually follows the convex bank where the tendency to refill is least. It is cheaper to construct, as the bank can be protected at less cost than a dike can be built; quicker, as the first cut of the dredge can be placed on the bank without using scows or barges for removing the excavated material; and better, as the channel can be straightened if needed and the minor irregularities in the bank corrected. Furthermore, spurs or training walls on both banks on opposite sides to contract the channel have been found difficult to navigate at intermediate stages of the river and often dangerous from the "draw" over them when submerged.

Second, the normal profile low-water width is calculated by the usual formulas involving the width, depth, and character of bed, the discharge and slope of the river, and when obtained, is checked with actual conditions in normal reaches nearby. Formulas are never entirely reliable, as no mathematical expression is equally applicable to the many varying cases met with, but they serve as a guide and when compared with the natural conditions are often useful.

Third, a combined system of bank protection, spurs and training walls, and sometimes sills when needed, are applied, so arranged as to hold up the water surface after the excavation is made, and so designed as to smooth out the inequalities in the low-water slope and distribute the fall over a longer stretch of river than before in order to reduce the extreme velocities.

Fourth, the bottom is drilled and blasted, if necessary; the material is dredged out and placed in the dikes, and after completion the shoal is re-surveyed to see whether further changes are necessary in adapting theory to the problem. Gauges are placed before

Fig. 4 (upper). Preparing foundation bed for navigable pass, Dam 28, Ohio River, November 12, 1912. Notice 5-foot by 6-foot cut-off wall at upstream edge of dam with two rows of 1½-inch twisted steel rods in bottom. Also three rows of same size rods spaced 4 feet center to center below cut-off wall. Rock is sound gray sandstone, except near middle of picture, where sound rock was 5 to 10 feet lower than on either side.

Fig. 5 (lower). Navigable pass raised, Dam 37, Ohio River; water about 18 inches below top of Chanoine wickets.

commencement along the site of the shoals, at the head and foot, and about a half mile above and a similar distance below, to ascertain the effect of the dikes. These gauges are read daily. When finished, survey of the work is made and the new slopes and velocities plotted. It sometimes happens that this survey will indicate that some dikes must be prolonged, or the channel narrowed, or perhaps a dike lowered in height to obtain the best results.

A very satisfactory example of this kind of work on the Tennessee River is at Little River Shoals. (See map herewith.) The maximum slope was originally 9.5 feet per mile at low water and is now about 5.3 feet. The low water discharge is about 2,500 cubic feet per second and high water discharge 395,000 cubic feet per second. The maximum velocity at low water was over 8 feet per second, and this has been so reduced that ''warping'' in passing up is no longer necessary.

On this work the rock excavation cost \$1.92 per cubic yard, made up of drilling and blasting at \$1.71 per cubic yard and rock dredging at \$0.21 per cubic yard. Gravel cost \$0.10 per cubic yard to excavate. The dikes cost \$1.74 per linear foot, or \$2.55 per cubic yard. Dikes were largely constructed of material excavated from the channel.

On the other hand, there are examples of a different kind of combination of dredging and contraction in the lower reach of the Tennessee River where the excavation is the predominating part and the contracting works are mainly for the purpose of compensating for the increased cross sectional area of the river, due to the newly dredged channel. In this section the distance from Riverton to Paducah is 266 miles; the total fall at low water, 77 feet, or an average of 0.34 foot per mile at low water. The low water discharge is about 10,000 cubic feet per second. The project provides for a channel 150 feet wide and 5 feet deep at low water over all the shoals. Dredging has been very successful on this section, and the dredged cuts remain open in most cases without any later work. This is believed to be largely due to the contraction of the river opposite each cut by dikes which are composed of the dredged gravel and so placed as to hold the water surface without material change. It has not been found practicable to raise this surface to any important degree by these dikes, but their apparent usefulness in preventing any decrease of depth warrants some additional cost for construction.

The various combinations of these two kinds of work, viz, contraction and dredging, represent one important method of improvement that has been extensively applied in cases where the river characteristics and required navigable depths permit, but their usefulness is by no means universal.

Canalization must be resorted to whenever the discharge of the stream is too small for open river work, or the slope too steep, or the depths attainable thereby insufficient, and will usually be considered when it has been found that the simpler and cheaper

Fig. 6. Placing horses and props for Chanoine wickets in navigable pass, Dam 28, Ohio River, December 29, 1912. Putting in concrete in deep hole at center where good rock was 5 to 10 feet lower than at either side. (See, also, fig. 4 for same spot about seventeen weeks earlier.)

means before described are not applicable. Canalization costs more per mile, its maintenance is higher, and its delays to downstream traffic are usually greater. But its results are positive and immediate; it renders upstream navigation easier than before, and affords greater safety to vessels than the other methods. Where downstream navigation is of great importance during high water stages, as on the Ohio, movable dams must be used, so that when dropped the river may be practically unobstructed at such stages. Such dams have greatly widened the applicability of canalization. A comparison of the cost of regulation and canalization is given below:

TABLE A.—*Cost of Regulation.*

River.	Depth, feet.	Miles.	Cost.	Average cost per mile.	Average fall per mi. in ft.
In the United States. (Western Soc. C. E., Feb., 1909.)					
Upper Mississippi, St. Paul to Missouri River	4.5	632	10,252,653.66	$16,222	0.44
Hiwassee	2.5	19	$74,726.06	3,933	0.93
*France. (British Waterways Com'n.)**					
Rhone (Lyons to Sea)	4.1	205	$13,445,000 (about)	$65,000	2.5
*Germany. (British Waterways Com'n.)**					
Rhine	4—10 3—4.1	214	$13,182,000	$61,000	1.14
Weser (near mouth)	5—10 3.6—4.6	210	2,316,500	11,050	1.65
Elbe (near mouth)	6—10 2.6—4.25	252	10,239,500	40,650	0.98
Oder (near mouth)	5—8.7	337	6,148,000	18,250	1.29
Worthe	3.3—4.0 3.3—5.0	215	2,509,000	11,675	0.81
Vistula	5—23	176	24,729,000	140,500	0.84
Pregel	----	78	599,000	7,700	----
Memel	5.25—6.6	69	2,902,500	42,100	0.48
Average, Germany	----	----	----	41,690	----

*£=$5.00.

TABLE B.—*Cost of Canalization.*

River.	Length Canalized Part.	No. of Locks.	Depth, feet.	Total cost.	Cost per mile.	Average length of pool.	Cost per reach.	Operation and maintenance. Cost.	Operation and maintenance. Cost per mile.	Operation and maintenance. Cost per lock.	Fall per mile in feet.
France (British Waterways Com'n).											
Saône	232	30	8.2	$8,765,000	$37,750	7.7	$292,000	$60,500	$261	$2,017	0.85
Seine, Montereau to Paris	61	12		4,955,000	81,500	5.1	413,000	54,350	891	4,528	0.85
Seine, Paris to Rouen	145	9	10.5	17,500,000	121,000	16.6	1,945,000	92,250	636	10,250	
Seine, New works	140	9	*	12,675,000	90,500	15.5	1,410,000				
Yonne	67	26		5,590,000	83,500	2.6	215,000	33,650	502	1,294	2.43
Marne	113.5	19	7.25	5,235,000	46,250	6.0	275,000	39,900	352	2,100	1.16
Aisne	35.5	7		970,000	27,350	5.1	138,500	10,650	300	1,521	
Scarpe	5	2		775,000	155,000	2.5	387,500	5,000	1,000	2,500	
Average, France					80,355		634,500	42,329	563	3,459	
Germany (British Waterways Com'n).											
Saar	19.5	6	4.25-7.9	$1,772,000	$90,850	3.2	$295,300	$32,050	$1,650	$5,350	2.15
Main	23.5	5	5.9-8.9	2,241,000	95,350	4.7	448,200	40,050	1,700	8,000	1.40
Fulda	17.0	7	6.6-11.9	785,000	46,200	2.4	112,150	21,925	1,300	3,125	3.30
Salle	89.5	15	4.4-8.8	1,917,000	21,400	6.0	127,800	45,400	500	3,025	1.27
Unstrut	40.5	12	2.5-8.5	528,000	13,000	3.4	44,000	14,200	350	1,175	1.72
Oder	53.2	14		6,059,000	113,900	3.8	432,800	217,100	4,075	17,000	1.90
Average, Germany					63,350		243,375	61,787	1,596	6,279	

*Over 10.5 feet up to Paris.

TABLE B (*continued*).—*Cost of Canalization in the United States.*
(U. S. Inland Waterways Commission, 1908)

River.	Length Canalized Part.	No. of locks.	Depth, feet	Total cost to U.S. up to 1907.	Cost per mile.	Average length of pool.	Cost per reach.	Operation and maintenance,‖ 1909.			Fall per mile in feet.
								Cost.	Cost per mile.	Cost per lock.	
Black Warrior	91	7	6	$2,540,397	$27,916	13.0	$362,914	$126,034	$1,385	$9,700	2.76
Coosa	25	3	3	1,048,438	41,933	8.3	349,479	7,934	317	2,645	2.58
Allegheny	25	3	-------	1,337,869	53,514	8.3	445,956	46,639	1,865	15,546	2.06
Monongahela	131	15	7–8	6,845,857	52,258	8.7	456,390	173,364	1,323	11,557	2.04¶
Muskingum	84	10	6	1,837,625*	21,876	8.4	183,762*	50,130	597	5,013	1.55
Little Kanawha	48	5	4	388,020†	--------	9.6	77,604†	8,040	167	1,608	1.23**
Great Kanawha	90	10	6	4,223,830	46,970	9.0	422,383	84,315	937	8,431	0.84
Big Sandy	27	3	6	1,205,954‡	44,665	9.0	401,985‡	15,848	586	5,016	1.43
Kentucky	226	11	6	2,903,309	12,890	20.5	263,846	159,644	706	14,513	0.87
Green and Barren	190	7	5	1,638,410	8,623	27.1	234,059	75,884	399	10,840	0.49
Cumberland	131.4	8	6	2,625,380§	19,980	18.5	328,172§	33,055	252	5,509	0.67
Illinois	194	2	7	1,515,721	7,813	48.5	757,861	14,840	765	7,420	0.05
Average in U. S.	-------	-------	-------	------------	30,762	--------	357,034	--------	775	8,149	----

*Repairs only.
†Operating cost only.
‡Includes operation.
§Office records.
¶ In West Virginia.
**About.
‖Annual Report Chief of Engineers, U. S. A., 1910.

From the foregoing tables it is seen that canalization is usually much more expensive to install than regulation, and if we add to its first cost the amount which represents the capitalized maintenance charges, the disparity will really be greater. But it is much less limited in its application than regulation, affords greater facilities for navigation, and is indispensable in certain cases. Its range of usefulness practically commences where that of open river work ends, leaving a comparatively small debatable ground between.

But with lateral canals the case is somewhat different. By means of a lateral canal with one or more locks, the obstruction in the river at any particular locality may often be obviated, and some years ago this method of river improvement was widely advocated. Brindley, an English canal engineer, is said to have stated that "rivers were created for the purpose of feeding canals," and apparently acted on this belief.

Lateral canals are expensive to build and maintain, frequently cause undue delay to vessels in passing each other or in grounding, are easily blockaded through accident, and require constant expensive attention. They form pools of still water where sediment collects, requiring constant dredging. At Des Moines Canal on the Mississippi River there had been removed from the canal between 1878 and 1906, 2,181,743 cubic yards of sediment, a volume larger than the original excavation.

At Cascades Canal, Columbia River, periodical dredging is necessary to keep the upper and lower entrances clear, and at the Muscle Shoals Canal, Tennessee River, one dredge is kept constantly busy through a large part of the year removing sediment brought in by the drainage of side creeks and the inflow at the head during times when the river carries silt. Since opening the canal (1892) over 1,300,000 cubic yards have been removed.

The costs of various lateral canals in this country and abroad are given in the following table.

TABLE C.—*Lateral canals.*

Location.	Length, miles.	Locks.	Cost.	Cost per mile.	Cost per reach.	Operation and care.	
						Cost.	Cost per mile.
France.							
Loire, Digion to Briare	121.7	37	$10,355,000	$85,000	$280,000	$31,650	
Garonne	120	53	12,420,000	103,500	235,000	37,500	$312.50
United States.							
St. Marys Falls	1.1	2	$8,057,252	$7,324,775	$8,057,252	$103,096	$93,723.00
Des Moines Rapids	8	3	1,583,046	197,880	791,523	38,772	4,846.00
Muscle Shoals	18	11	3,191,726	177,318	290,157	51,420	2,856.00
Colbert Shoals	8	1	2,207,941	275,993	2,207,941		
Cascades Canal	½	2	3,820,325	7,640,650	1,910,162	14,379	28,758.00

The high first cost and later charges for maintenance make this method of improvement such a costly one that it would probably to-day be often supplanted by canalization. High speed can not be allowed in canals owing to the wash caused along the banks, but in canalized rivers no such limiting condition exists. Col. Wm. E. Merrill, Corps of Engineers, an eminent river engineer, stated in 1887, while commenting on the Muskingum River, as follows:

"The greatest obstruction to successful navigation on the Muskingum is caused by the lateral canals; they are expensive to keep up on account of the guard gates, the numerous drawbridges, and the necessity of periodical dredging. Even when in perfect condition they must be navigated slowly, and it is with difficulty that boats can pass each other."

At Colbert Shoals on the Tennessee River the lateral canal now nearly completed provides a lift of 26 feet, which is overcome by a single lock. Before undertaking its construction it was several times proposed to build one or more dams across the river and canalize these shoals, but the possible use of the river at high stages, when fixed dams might interfere with boats, decided the engineers to adhere to their first selection of a lateral canal.

At the Des Moines Canal a project is now on foot to build a power dam in the Mississippi which will "drown out" the present canal. Locks will be used in passing vessels from one pool to the other, but the old canal will have to be abandoned should this plan be put into execution.

Similar plans for the Muscle Shoals Canal on the Tennessee River have been proposed, but no satisfactory agreement has ever been made between the power corporation having the work in view and the government as to the proportionate share each should pay toward the project. As a consequence, nothing has yet been accomplished.

These instances are mentioned to show that the conditions affecting the selection of this type of work are changing and to-day lateral canals would probably not be selected in some of these instances.

In general applicability canalization is easily first among methods and is now being used more than all the others combined. But little new work of regulation is being undertaken anywhere and no new lateral canals are now being commenced in this country. Except the Rhône and Loire, nearly all the improved rivers of France are canalized, about 970 miles in all; and in Germany, where there are nearly five times as many miles of open as canalized rivers and regulation finds its warmest advocates, but little new regulation work of importance has been done since 1875. In Belgium there are four times as many miles of canalized rivers as open rivers. (Royal Commission, Waterways and Canals.—Lindley.)

A fair conclusion is that the present tendency is toward the selection of canalization whenever a new system is to be chosen, par-

ticularly if the examination of the natural and economic conditions shows regulation to be inapplicable. It also seems that lateral canals are not now being regarded favorably unless the local conditions unequivocally indicate the necessity of their adoption.

The theory of the use of reservoirs as an exclusive mode of improving rivers for navigation is an old one practically abandoned now in this country by river engineers but revived from time to time by its advocates and sometimes made the subject of an academic discussion of considerable seriousness. At first glance the theory is very engaging, but experience in its application has been in the main unsatisfactory and its applicability depends on many assumptions which are themselves of doubtful tenability.

The possibility of conserving the high water flow of rivers, thus reducing injurious floods and later using the stored supply during the low stages for the benefit of navigation, is fascinating alike to layman and theorist and offers a wide field for speculation. But the attempt at practical application usually introduces virtually insurmountable difficulties.

The theory takes it for granted that sites for reservoirs, ample in number and capacity, can be obtained without greater injury to railroads, factories, towns, and valuable private possessions than the benefits to be obtained for the public; that enough high dams can be safely built at reasonable cost free from danger of a breach, operated intelligently and efficiently in order to create the necessary storage space for regulating the discharge. It assumes further that these reservoirs are not likely to be soon filled with silt but are practically of permanent usefulness. It assumes that increased discharge in the rivers means increased depths, and that precipitation may be foretold with sufficient accuracy to permit successful regulation. All of these assumptions are more or less debatable and some of them are plainly of doubtful reliability.

The reservoir theory has never been practically applied on a large scale as an aid to navigation except in two cases, and the results there have not been encouraging. It was early adopted on the headwaters of the Msta and Volga rivers in Russia, where its use was the outgrowth of very favorable natural conditions. These rivers, flowing in opposite directions, take their source in a lake and swamp region at an elevation of 665 feet above sea level where land was cheap and unfit for agriculture. Here the construction of low, cheap dams was an easy task, resulting in increasing the reservoir effect very greatly without the necessity of obtaining new storage sites or going to much expense for dams and regulating works. A natural reservoir system was in fact already in operation, needing only a little artificial regulation. The capacity of this system is 35,000 million cubic feet with dams only 17.5 feet high. (Reservoir sites in Wyoming and Colorado.—Chittenden.) It is usually considered a fairly efficient work in lengthening the season of navigability of the two streams.

The same favorable natural conditions were found on the upper

Mississippi River and similar reservoirs were built by the government for improving navigation in the 70's. By constructing low dams across the outlet of an extensive lake area, a storage of 93,400 million cubic feet was accomplished at a total cost of only $678,300. (Reservoir sites in Wyoming and Colorado.—Chittenden.)

Although the Russian reservoir system was in a measure successful, the effect of these reservoirs upon the Mississippi River at St. Paul, 357 miles below the dams, was slight, being from 12 to 14 inches on an average, and 51 miles further down all trace of it disappeared altogether. But the dams were so beneficial for the production of power that when their abandonment by the government was being considered some years later such a course was vigorously opposed by the milling companies and other similar interests benefited, although they contributed practically nothing toward the expense of the work.

The system has never been extended in the United States, although often studied. The slight assistance to navigation and the high comparative cost of construction and maintenance were reasons given for not applying this method of improvement to the St. Croix, Chippewa, and Wisconsin rivers, which were examined several times a few years later by engineer boards. (Annual Report, Chief of Engineers, 1887, page 1692.)

It is stated that early in the last century the plan was adopted to some extent in France for the control of floods, and in 1856, after a flood of unusual destructiveness, it was thoroughly studied, more particularly with reference to its applicability to the Rhône, Seine, Garonne, and Loire. As a result of this investigation, it was decided not to construct the reservoirs proposed for these streams, owing to the "uncertainty and doubtful efficacy of their action." In 1881 this system was definitely abandoned by the Corps des Ponts et Chaussées, and its use for the control of rivers condemned in France. (Annales des Ponts et Chaussées, 6 sem., Vol. II, 1881.)

The plan was proposed for the improvement of the Ohio in 1873 but was not favorably considered after careful investigation. The board in whose hands this investigation was placed stated that "The first of these plans (viz, storage reservoirs) the board deems impracticable on account of the difficulty, if not impossibility, of finding locations for the necessary reservoirs, the immense cost of the system, its interference with nvigation of the tributaries on which the dams are located, its injury to agricultural, mining, and railroad interests in the valleys of these rivers, the difficulty of regulating the supply from the reservoirs, and the terrible effects that could be caused by accidents." (Annual Report, Chief of Engineers, 1873, p. 541.)

In 1909, the plan was again suggested to a board of engineers having in their charge the project for the improvement of the Mississippi River, and again the plan was abandoned for lack of suffi-

cient reservoir sites, high cost, and uncertainty of action. The board stated in its report that "In order to use this reservoir system for the benefit of the improvement of the river below St. Louis it would be necessary to commence the discharge at the reservoirs at least two months before it was needed at St. Louis, and a still greater interim would be necessary for the benefit of the improvement of the river below Cairo. Experience does not justify such long forecasts, and the service of the reservoirs would necessarily have to be based on general annual averages, an unreliable and unsatisfactory basis." The board also stated that "There is no instance on record where this system has been applied with benefit commensurate with the expense." (House Ex. Doc. 50, 61st Congress, 1st session, p. 17.)

More recently, while a project for the improvement of the Ohio River was being considered by a board of river engineers, a reservoir plan was brought forward by some officials of the U. S. Geological Survey who urged its adoption. Their preliminary estimate of cost of the reservoir project was given at $125,219,000 (*Engineering News*, May 7, 1908), which was later admitted to be much too small. It was found on more detailed examination that the cost would likely be nearer ten times this amount. (*Engineering News*, Oct. 8, 1908.) The enormous cost of the reservoir plan and the uncertainty as to its successful operation, combined with its unsuitability to the topography of the Ohio River valley, were reasons for its rejection and for the selection of the cheaper and more certain method of improvement by canalization, using movable dams. The estimated cost of the adopted plan for 9-foot depth by locks and movable dams is about 63¾ million dollars.

The general impression among river engineers in America seems to be that storage dams for the benefit of navigation alone will never be warranted. Similar dams have been constructed in many places for industrial purposes, such as power development and irrigation; but these purposes are not always in harmony with channel improvement, and the incidental benefits likely to be received on navigable streams from dams built for several combined purposes can not always be determined in advance nor their value accurately estimated.

If an added flow at low water be furnished from such dams, the valuable scouring effect of low water may not be obtained without a supplemental series of contraction works at further additional cost, and the increased discharge may not mean increased depths on many sediment bearing streams. For example, on the Mississippi where much material is rolled along the bottom, bars often rise and fall with gauge heights, the low water being largely relied on to restore the channels in such cases.

Then, too, the location of the dams on the tributaries would usually be such as to intercept much of the flow of silt in suspension in the portion of the stream where the scour is greatest. In no other way could the water be clarified. This clarification is an

assumed advantage of the reservoir system that has often been mentioned.

There is no known way of safely and easily removing silt from behind storage dams, more particularly if it is expected to do so without much injury to the river channels below. A constant diminution in storage capacity would be one of the inevitable results of the system, or an injurious deposit in the lower channels it is intended to benefit.

Notwithstanding these disadvantages there may occur special cases where some incidental benefit may be derived, but experience seems to point out that such benefit will hardly ever be sufficient to very strongly influence the location of any storage dams or warrant any considerable portion of the cost being borne by the navigation interests.

As a summary of this discussion, the following conclusions are briefly stated:

1. Regulation in some suitable combination with channel excavation should always be first studied as a method of river improvement and adopted in all cases where economically applicable. It will be oftenest used wherever the funds available are small in amount, the increase of depth needed not great, the river flow comparatively large, the banks low and the width of the river considerable, velocities low and regimen more or less fixed.

2. Canalization with movable or fixed dams will be adopted wherever regulation with channel excavation is insufficient or unsuitable. It will usually be applied where the slope is steep, discharge small, and depths obtainable by regulation insufficient.

3. Lateral canals should never be selected for use unless imperatively demanded by the local conditions.

4. Reservoirs are too uncertain, too unsafe, and too expensive for exclusive use in river improvement. They will seldom be relied on, except in special cases in connection with other enterprises where their use for industrial purposes warrants the cost and the water flow can be sufficiently controlled to operate beneficially on the channels.

The Fifth Arm

A great deal is being written at the present time about the value of air craft in future wars. The enthusiasts are inclined to claim that man's ability to navigate the air will completely revolutionize warfare. The sober student, however, of military history and military operations in general does not agree to this. Nevertheless, air craft (aeroplanes and dirigible balloons) are going to exercise a very important influence in all future wars between civilized nations. The following extracts from two different periodicals furnish food for thought for the engineer, the artilleryman, the cavalryman, and the infantryman:

THE HIGHWAY OF THE AIR AND ITS MILITARY ENGINEERING PROBLEMS*

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To the military engineer dealing with fortifications (earthworks and obstacles), communications (roads and bridges), and with mines and demolitions, the flying machine particularly beckons, as the change of conditions will mean to him a reconsideration of present methods and ideas.

The New Material.

The flying machines of the present day are either gas supported (dirigibles) or heavier than air (aeroplanes). The former are either "rigid" (having internal solid skeleton) like the *Mayfly* and the *Zeppelin*, or "non rigid," like the *Parseval*, *Gross*, and other military balloons.

The rigid dirigible has a speed of 30 miles per hour, with a range of 1,000 miles, carrying fifteen passengers without descending; though this was under most favorable weather conditions and not to be expected during hostilities.

It is unreliable in windy weather, and the fact of so many being storm destroyed proves it must be protected by a kennel; and, furthermore, the difficulty of emerging or entering its house during wind necessitates that the kennel be turntable, either mechanically

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*By Lieut. G. A. Taylor, Australian Intelligence Corps, New South Wales. Extracts taken from the *Commonwealth Military Journal*, Melbourne, Australia, for March, 1912.

or by placing the same on water; hence it must always act from a base. The "non rigid," however, can be deflated, it is thus more transportable; and where railways are available its gas plant can be brought near the arena of operations.

But the value of a military weapon depends on its being utilized under all conditions in which an enemy can operate, and in this respect the aeroplane comes nearer the military ideal. It can travel horizontally twice as fast and rise three times as quickly as the dirigible; a vital factor in aerial tactics; and certain types can be flown in winds up to 40 miles an hour.

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How Aerial Tactics will Affect Military Engineering.

The military engineer, in considering aerial tactics, will find for reconnaissance purposes the aeroplane provides a new field of observation. It has advantage over mounted patrols, inasmuch as it covers country at a greater speed; but it has the disadvantage of breaking the great commandment of true reconnaissance, that is, "See without being seen."

The aeroplane, I noted, during the recent flight at Liverpool Military Camp, could be observed 5 miles off, though the camp must have been visible to it from a farther distance. From a height of a thousand feet, a distance of 33 miles can be seen; but for accurate reconnaissance, and no other is worth considering, closer approach must be made; hence reconnaissance information from an aeroplane, to be of any value, must be instantly transmitted.

If the aeroplane has to return and maneuver to safety, land, and deliver the message, the enemy would have time to change its formation, and in that respect new problems in strategy will engage military engineers; and Stonewall Jackson's advice to "always mystify and mislead the enemy" will take a wider meaning.

Trap formations will be made on even grander scales than Napoleon's formation trap at Austerlitz. It is well known, yet its repetition is worth while.

He took up a position behind cover with his left flank touching the northern of two ranges, and a great gap between his right and the southern range. Through this gap it would seem easy to penetrate and so turn his right flank and cut his communications. His enemies fell into the trap. Napoleon attacked their right wing to let the left get well away on its great outflanking movement, he then threw 30,000 men at their weakened center, seized their strong tactical point, cut the army in two, and smashed nearly half of it.

The military engineer will have to be prepared for many such schemes of deception and surprise.

In the first battle of wits in aeroplane military tactics, the ground engineer won out over his aerial brother. By painted tree trunks and dummy earthworks, the reconnoitering aeroplanes in German maneuvers (1909) were hoodwinked, returned to their base with misleading information, their army falling into the trap.

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By the most mobile of the forces will the information be turned to best account, so to the advanced mounted troops will the reconnoitering aeroplanes be allotted. The aeroplanes, however, can not give the best indication of tactical strength—an aeroplane observer, for instance, can not say if a house is occupied, hence negative information from the air is unreliable. A forest may screen an army, yet a body of light horse could go through that forest like a rake.

The aeroplane will precede the advanced patrols, giving the most advanced strategical information, yet it must be remembered that aerial observation from a height is at times misleading. As one rises from the ground, the horizon rises also, the ground below sinking, the country forming, as it were, a cup, hills first appear levelled, and then are almost unobserved except in sunlight, being then distinguished by their shadows. Aerial reconnaissance from a height practically only gives horizontal plan—vertical elevation must be obtained by low flying or by mounted troops.

Reconnaissance from a height is at times misleading. Take, for instance, a view from the top of a monastery at Kensington, 250 feet high, looking towards an enemy occupying a ridge between Long Bay Penitentiary and Botany Bay.

The foreground from that height corresponds to the view of an aeroplane observer a mile off at a vertical height of 1,000 feet, the ground appears fairly flat, without any cover worth considering, and any kind of a frontal attack seems absurd. A close ground study, however, reveals a deep gully running well up towards the position, and offering splendid cover for infantry, with opportunity for a frontal attack in extended order through the scrub, whilst the enemy's flanks could be kept active.

The aerial engineer, for reconnaissance, will have to fly low and take risks.

In naval tactics, the aeroplane will precede the destroyers, locating the enemy's submarines, and acting as advanced aerial patrols.

Aerial Attack.

In attack from the air, the military engineer will have to consider possibilities of aerial bombardment. Both dirigibles and aeroplanes can be utilized for aerial attack—bombardment from dirigibles being the more accurate and having greater effect, for, being stationary, there is best chance of accuracy, and heavier bombs may be carried; but with an aeroplane, given the height by aneroid, speed by anemometer and horizontal distance, accuracy of hits is but a simple matter of the parallelogram of forces, and a target can be quickly picked up by running fire from a bomb dropper (quickly worked by foot pressure) like an aerial machine gun.

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The value of sudden aerial attack is very great. Surprise has been the greatest factor in almost all the grand strategical combi-

nations of military history. The first and last thought of every great general is to strike where he is least expected, and it is through such possible surprises that the engineer in the past, with his field defences, has nursed the fighting spirit of the soldier, and kept at a distance that spirit of demoralization that stalks every army, that spirit that watches every opportunity to wreck the moral force of the fighter, for more armies have been routed by moral effect than by shot and steel.

It was the moral effect of Jackson's counterstroke that won the battle of Bull Run: "Reserve your fire till they come within 50 yards, then fire and give them the bayonet, and when you charge yell like furies," and that yell rang in the ears of 12,000 Federals and chased them to Washington.

It was the moral effect of Napoleon's tactics that won the Battle of Ulm, when 23,000 surrendered without firing a shot, when Napoleon said: "My army can fight with its legs."

Consider the moral effect of a screaming aeroplane shooting into and scattering the ranks of an army, marking its track with explosives. Consider the effect the possession of swiftly moving airships will have upon an army not so armed, who would watch with fear each hovering cloud to see if it screened the aerial terror that was smashing them with smokeless noiseless guns!

It will be to counteract in some degree the moral effect of aerial attack that the engineer will have some problems to solve.

Aerial Defence.

Aerial defence may be summed up in excess of speed—horizontal and vertical, and the use of smoke, cloud, fog or mist cover. Organization must be perfect; and aerial engineers, pilots, observers, and mechanics must be well trained in the use and possibilities of all arms.

Ground Attack.

The attack from the ground will take the form of high angle fire, and in this respect the engineer will consider the best means of assisting field guns to rapidly allow for swift aerial movements, by devising special elevating carriages. The elevation of present mountain guns is but 15 to 30 degrees, howitzers, 45 degrees, and the new Krupp gun 70 degrees.

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High explosive shells will have a very marked effect exploding in the locality of an aeroplane. The air concussion of gun fire is sufficient, as is known, to break windows in the vicinity, the concussion effect of the shell will, therefore, be much greater in the air as there is no ground friction, and it will be in the study of the concussion effects of certain explosives that the engineer will have some problems to consider.

In this respect an interesting accident happened recently during the maneuvers on the Mexican frontier. Simon went out to locate

three batteries of artillery. He discovered two and flew over the third without locating it. As the artillery had orders to fire blank cartridges at any machine within range, they let go at Simon, who was flying at height of 800 feet. The air concussion almost upset him. Here was an instance of an ordinary field battery blank fire having an effective range of 1,600 feet diameter.

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In attack from the ground, much ammunition will be wasted, hence the engineer will facilitate mechanical arrangements for speedy supply. In late wars it has been estimated that 1,000 to 3,400 cartridges and 80 to 100 shrapnel shells have been expended for every man put out of action. The aerial attacker justifies still greater expenditure. It has, therefore, been allowed that 8,000 cartridges are justified for an aeroplane with one observer. Therefore, if one company can fire 1,500 rounds per minute, success in ground attack of aeroplanes will depend on the expenditure of much ammunition, and with several companies maintaining requisite rate of fire; sights of rifles and machine guns being altered to allow of aerial elevation and deflection.

Automobile guns are now in use; but having to keep to the roads are under a disadvantage in following aerial movements (see illustration), particularly as aerial attackers may seek cover in clouds, and under the best conditions, remain but a short while in range, and then suddenly change their direction on apprehending danger. And it is in this superiority of direction over roads and such present lines of communication that the aeroplane has the advantage. Take a road in the stress of war. It is the life line of the army, it holds it to its supplies, and along its well-tramped length rush the cheering reserves to replace losses—the food transports, the ammunition carts, hospital supplies, all rush ahead in a cheering, wildly excited line; but coming back there crawls another line of wounded soldiers, dejected prisoners, empty wagons—two opposite moving lines struggle along, hedged by the sides of that road; each has to make its way unimpeded or the army suffers. The protection of that road has been the main concern of the campaign; cut it ahead and the army is checked; cut it at the rear and the result may be more disastrous. The fate of a nation may depend on the break of such a line of communication. Napoleon made good roads his first necessity. In fact, till to-day "the road" has been the fulcrum upon which the fates of armies have been balanced. Yes, till to-day, for the aeroplane knows no road, and so will be difficult to follow up in ground attack.

It, however, must alight to replenish its store of petrol and oil, and when down, it will be like a tortoise on its back.

The alighting base, therefore, will give its position away, and it will be rushed by the mounted troops, and petrol stores located. The military engineer will, therefore, devise overhead and hori-

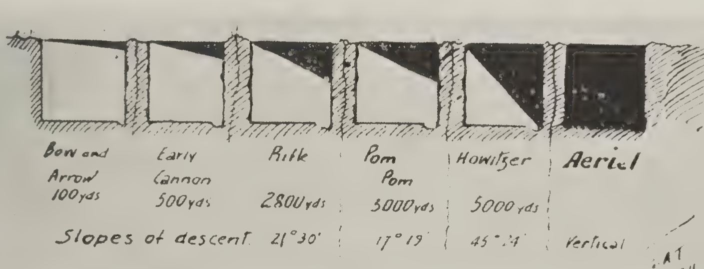
zontal cover for stores, and see that they are not concentrated, but well scattered—before they are scattered by an aerial visitor.

Ground Defence.

It is in defence from aerial attack that the military engineer will find some of his greatest problems.

The first principle is that field defences be so constructed as to conform with the tactical plan of operations, and as the tactical plan of operations of aerial attack means sudden surprises, quick arrangements for overhead screen and cover must be considered.

A TYPICAL ROAD IN THE AIR AGE.
(The excellent overhead screen will be noted.)



ZONE OF RISKS IN TRENCHES FROM ARMS ANCIENT AND MODERN.

Fig. 1 (top); fig. 2 (bottom).

Hitherto the weapons reckoned with in devising works of defence have been rifles, machine, and quick-firing guns, heavy artillery and howitzers.

Looking through development of modern weapons, we find field defences have been gradually getting more unsafe.

Of rifles, the slope of descent is 1 in 2½ at 2,800 yards. The greatest slope of shrapnel is 1 in 4 at 4,500 yards. The greatest slope of howitzers is 1 in 1 until with aerial bombardment we reach the vertical. * * *

I have heard military officers say that the dropping of explosives

from aerial machines can be prohibited by a Peace Conference; but the Hague Conference, 1907, refused to continue such prohibition, as it was felt that as aerial vessels would be used for reconnaissance, it would be unfair to deprive them of means of retaliation if attacked. Even if future rules of war prohibited dropping bombs, they could be fired down from guns of small carrying power.

Bombardment, however, is a wasteful method of getting results. The bombardment of Paris, though it practically used up all the German ammunition, only killed 96 people, and in an aeroplane ammunition is particularly limited, but the speed of the latter can enable it to quickly replenish.

The military engineer will, therefore, seize every opportunity to devise overhead screening and defensive cover, and no better screen will come to his hand than that devised by Nature since the beginning of things—trees.

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The advantage of trees as natural overhead screen was shown at Liverpool, where the Bristol machine flew over the bush unknown to the Light Horse squadrons maneuvering beneath.

Active steps will be taken to screen our roads with wide-spreading shade trees, and the military engineer, in times of peace, will be engaged in planting avenues of great and beautiful roadside shade trees, and be blessed by those who come after for bringing the lungs back to the earth again.

It will be remembered that during the Japanese War, Kuroki, under cover of night, transplanted a fir forest along the side of the road to Wiju, so that his army passed unobserved to the Russians on the hills beyond, an excellent example of tree screening from horizontal vision; but the roads of the future will be screened also from vertical observation. (See illustration.)

From the aerial attacker new cover will be devised, and the Japanese painted screens of the late war may yet be wisely imitated.

Infantry will study shadow throwing. In bright sunlight the shadow of a pole will screen a few, and the shaded side of a gutter will screen many.

Lying down shadows will be studied, lying in the direction of light giving least shadow, and so less chance of observation from the air.

Some experiments I tried in this respect from the Randwick wireless tower, though only 230 feet high, were really surprising.

Open country will, as far as possible, be avoided by mounted troops, except in the widest of extended order.

Speed and movement, like modern life, will quicken up warfare.

In bridge building rapidity will be the main point, and the type of bridge built over George's River during the recent maneuvers, will have a special value, being quickly constructed, having less

chance of being hit, and suiting troops in extended order. (See illustration.)

The Lessons of the Air Age.

Time will more than ever be everything. It will be annihilated with wireless telegraphy. Aeroplanes and advanced patrols will carry portable sets for quick transmission of messages and sketches. We shall see the last of those maneuvers with the flanks out of touch with their center, because visual signals had failed. Wireless will

Fig. 3. An air age bridge.

operate either in flat or bushy country where visual signalling is almost impossible, and during cloud, fog, or storm, when the helio is out of action.

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The coming age will be the age of initiative. The history of warfare has shown how initiative has extended through the forces. Time was when armies marched to war, each man a trustful unit doing only what he is told to do, and unable to do anything he had not been told—into war he has gone a pitiful fighting animal trust-

ing the initiative of his commander like the two mules of Frederick of Germany.

"See those two mules," he said, "they have been through twenty campaigns, and—they're still mules."

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Henderson points out that the initiative of German subordinate officers was the real cause of the swift destruction of the French army and the secret of German strength.

In the war of the future, this initiative will still further spread; in fact, every soldier will carry his entrenching tool.

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War will become the world's greatest science, and victory will remain with the nation best trained. The fighters of the future will not come from our football or cricket fields. The future Wellington will tell a different story than that of Eton playing fields.

The battles of the future will be fought in our universities and engineering colleges, and the nation best educated in that respect will win out in war as well as in peace.

To-day the soldier in almost every nation is the plaything of politics. His necessities are regulated by political exigencies. Defence estimates are butchered to build up financial statements.

The average layman is ever ready to put his opinion against that of the military expert.

The world denies to the highly trained military officer that specialism it readily recognizes in doctors and other scientific men; but in the coming age the public, in its utter helplessness against the aerial terror, will grimly recognize the importance of the military expert, and honor him accordingly.

MILITARY AIRCRAFT.*

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Flights to be Expected on Service.

There are very few cases of accidents due entirely to the wind, and a well-designed, strongly-built machine has probably very little to fear from the wind so long as it has room in which to recover. Starting and landing are the dangerous periods. There is, however, the physical fatigue of the pilot to be considered; some machines are heavy on hand, and I have seen pilots utterly exhausted after half-an-hour's fight with the wind. Cold is another factor; no amount of clothing seems to keep one warm on some days, and a pilot has very little control over a machine once he gets really numbed.

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*Extracts from a lecture given at the Staff College, October 26, 1912, by Maj. H. R. M. Brooke-Popham, Oxfordshire Light Infantry, Royal Flying Corps. Reprinted from *The Army Review*, London, January, 1913.

Dual control is now fitted to many aeroplanes, but nearly all pilots disconnect it before they do any flying. One will want to have great confidence in the other pilot before trusting oneself in the air in a dual-control machine.

During the first week of Cavalry Divisional Training this summer, the conditions were as bad as they were on any other week during the year, but there was no day on which one of our three machines did not cover a distance of at least 40 miles, and, of course, we should take far greater risks on service. But aircraft may have too choose their time of day, and we have not yet arrived at the stage of being able to order out a machine as one would a cavalry patrol. Also pilots can not fly continuously even under good weather conditions, and it is unsafe to count on a pilot flying more than ten hours out of three days.

Then we shall be lucky if we ever have more than half our machines in action at any one time on service, and even to do this we shall want a large number of spare parts and reserve machines. The French, I believe, intend to keep for every six aeroplanes at the front, two complete machines and three more without engines, instruments, or petrol tanks, the idea being to take these out of damaged machines. We must be careful to avoid drawing false conclusions from maneuvers; when an aeroplane comes down, then, inside the enemy's lines during a field day, all the pilot has to do is to telegraph to the field base; a car with mechanics and spare parts is sent out, and the aeroplane is flown back in the evening ready to go out the next day. But we can not do this in war; if anything goes wrong then, the pilot and observer will spend the next day *en route* for a prisoner's concentration camp.

An Italian officer, who has been flying in Tripoli, was quite certain that no man could go on flying in war for more than three months at a time; and they had no hostile aircraft to contend with.

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Judging by our experiences this year, the utility of aircraft in war is more likely to be limited by difficulties of observation owing to mist and low clouds than by inability to go up owing to the wind. It is as well to remember, however, that England is noted for fog, and so we are likely to find aircraft of even more value on the Continent than at home. On a clear day it is quite easy to carry out observation at a height of 3,000 to 4,000 feet, and this is better than lower down, because objects can be kept in view for a long time and so it will be easier to identify the exact positions of troops on the map. In misty weather, however, or when the clouds are low, we should have to come down and take our chance of being hit by fire from the ground.

At 3,000 feet on a clear day individual carts can be seen and horsed vehicles distinguished from motors. Individual squadrons and companies marching along a wide road can be seen 2 or 3 miles off, but troops on narrow roads with high hedges may escape observation. Artillery can be distinguished from transport by the

number of horses, and, as a rule, the gun itself can be seen. Bivouacs are very easy to see, especially if shelters of waterproof sheets are put up, and individual companies in bivouac can be found without difficulty. If a bivouac is disposed neatly, with nice regular intervals, it is a simple matter to count the number of units. Empty bivouacs can be recognized if the fires are left burning. Troops in billets are hard to find so long as the men stop inside the houses, but individual squadrons can be seen forming up in villages. Detrainments can be seen.

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Avoiding Observation from Aircraft.

To keep hidden from aircraft I suggest making use of low-lying roads, and taking advantage of the morning and evening mist. When halted, troops should get inside woods or houses, or close up against the hedges of the roads. If time is of less importance than concealment, it might be possible to have special men with whistles on the look-out for aircraft, and for everyone to halt and make for the nearest cover as soon as a danger signal was blown, but guns and transport will be a difficulty.

If forced to halt in open fields, all regular formation should be avoided and the field chosen so as to correspond with the color of the uniform. I do not think marching by night can be kept up for many days in succession owing to the fatigue caused to men and horses, nor does it seem practical to hide in a forest waiting for a storm.

Tricks may be practised on aerial observers, such as marching at large intervals so as to greatly exaggerate the length of a column, painting wagon covers and putting on an extra pair of horses to make transport look like guns, marching along one fork of a road for a little way in the evening and turning back along the other fork after dusk.

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Dirigibles.

I should like to say a word here about dirigibles, because we in this country are rather inclined to under-rate their value.

Observation from them is much easier than from aeroplanes; they can hover over any spot and can drop bombs with considerable accuracy. Athough they do not travel as fast as aeroplanes, they can carry a long range wireless equipment and so get their information back quicker. Their radius of action is much greater than that of aeroplanes, and, provided they can get out of their sheds, they can go up in as bad or worse weather.

On the first day of our maneuvers this year the *Gamma* was out on a four-hour reconnaissance at a height of over 4,000 feet, and was in wireless communication with headquarters the whole time, every message being received correctly. Airships may also be of great value by night, when they seem very hard to spot. Last

autumn, for instance, a French dirigible, *L'Adjutant Reau*, sailed over Verdun by night, and though the garrison of the fortress were on the look-out for her and had searchlights going, they failed to discover her till she had got out of range.

The latest German airships, which are about four times as long as our *Gamma*, can remain in the air for 2½ days, rise to 6,000 feet in 5 minutes or less, have a speed of 50 miles an hour, and can carry enough fuel to travel 1,500 miles at a reduced speed. They could substitute explosives for some of the petrol and oil, and drop at least 600 pounds of this at a time without upsetting their stability. Experiments have been tried with several different kinds of bombs, incendiary and otherwise, including some called by the expressive name of "stinkbomben." They carry wireless with a range of 250 miles, have two machine guns in the car, and an observer with another machine gun on the top of the envelope. I am afraid that such a vessel would not be so much at the mercy of an aeroplane as we are inclined to think. Germany has five of these under construction for her navy, and I believe two for the army. She also possesses ten of a slightly smaller size. France has just given orders for four vessels of much the same type.

I may add that Germany now possesses about 120 aeroplanes, all of them made in the country, except a few experimental ones. She will have nothing to do with French engines, but relies partly upon Austrian firms and chiefly on her own. The observers are systematically trained both in observation and in mechanics. Although Germany has produced no great flier, the average of skill is probably as good as that of the pilots who were flying on our maneuvers this year. The subscription fund in aid of German aviation now amounts to £300,000, that of France being about £120,000.

Aeroplanes in War.

It is doubtful if aeroplanes have much to fear from rifle fire if at a height of 3,500 feet, and if a chance bullet does hit a machine it will not necessarily disable it. An Italian officer in Tripoli had his machine hit on seven different occasions while at a height of 2,000 to 2,500 feet, but it was never disabled. Bullets, curiously enough, seem to have but little effect on a propeller.

Several types of anti-aircraft guns have been made. Germany has some mounted in motor cars, which can be elevated to 72 degrees and have a range of 10,000 yards horizontally and 6,500 yards vertically. The shell leaves a smoke trail.

Whether fire from the ground is effective or not, it is inconceivable that the army which has the better pilots and more numerous machines will rest content with a mere exchange of information and not make serious attempts to destroy the hostile aircraft with their own. Fighting in the air will undoubtedly take place, in all probability with firearms. Ramming seems almost too drastic a method of procedure, and although one aeroplane may upset another by the backwash from its propeller, it will probably not

get to sufficiently close quarters if the other machine carries a man with a gun. There are several types of aeroplane already in which the observer could use a rifle or automatic pistol.

* * * * * * *

* * * Suppose the enemy tries to cover some particular area, of concentration, for instance. If he keeps aeroplanes on the ground they will take a long time to get up to 3,000 feet—15 minutes at any rate—and by that time our scouting machine should have seen all it wants to, and if it has superior speed it can get clear away. Airships can get up quicker, but our aeroplane, given superior speed and hardiness, could keep out of range and still do its reconnaissance.

Can the enemy keep his aircraft up in the air and form a defensive ring? An enormous number will be required; 20 miles square is not a very big concentration area, and that means a perimeter of 80 miles, for, of course, it is no good merely putting out a line in front.

The machine required for long distance reconnaissance seems then to be one with a speed of perhaps 90 miles an hour and a radius of action of 300 miles.

Things will be somewhat different as the hostile armies approach each other. Collisions between hostile aircraft will become more frequent, and the type of machine required will not be the same. It need not have such a large radius of action, 200 miles say; it can be content with a slower speed, say 70 miles an hour; but it must be armed and able to use its weapon efficiently. This will mean the propeller behind and a man in front with a firearm, perhaps a rifle, perhaps a machine gun, possibly some special weapon.

It seems probable that there will be a succession of duels between machines, or perhaps pairs of machines, when the opposing armies are within two or three days' march of each other, and that command of the air will result as the cumulative effect of a series of victories in such combats. Once gained, command of the air will be complete, not so much from the destruction of hostile machines as by the effect on the nerves of the surviving pilots.

We thus arrive at two main types of military aeroplanes, the scout and the fighting machine, and this seems a more logical grouping than biplanes and monoplanes, or single seaters and passenger carriers.

What the future may have in store it is impossible to say, but one type can not do both duties as yet; we can not combine speed, endurance, and fighting qualities, chiefly owing to the want of a good high-powered engine.

Possibly a third type may be required for intercommunication, a light quick-rising machine capable of landing on bad and restricted ground.

* * * * * * *

There seems no satisfactory method of distinguishing hostile from friendly aeroplanes. Colors can not be distinguished from

the ground if the machines are over 2,000 feet. There seems, however, a tendency to develop along national lines, and probably a trained man could distinguish aircraft much in the same way a sailor can distinguish ships. Then, of course, there are occasions when secrecy is all-important; when our aircraft would not be used and all seen treated as hostile. On August 16, 1870, it would have done Alvensleben little good to have known every detail of the French disposition, but it was of the greatest importance for him to conceal his weakness.

* * * * * * *

Aeroplanes and Cavalry.

Aircraft are of little use in fog, darkness, or stormy weather. Troops may escape the notice of aerial observers, or we may lose command of the air. Aircraft will certainly not render cavalry useless even for reconnaissance. But they will save the cavalry much waste of energy by doing most of the long distance reconnaissance to the front or flanks and in preventing their being sent out on useless missions searching for the enemy where none exist. There can be no question of aircraft *versus* cavalry, but only of aircraft and cavalry, each supplementing the other, and the former enabling the latter to carry out its work to greater advantage.

There was a good instance of the combination of cavalry and aircraft on the second day of maneuvers this year. The bivouacs of the 2nd Red Division were not located, but the cavalry found their outpost line and an aeroplane saw all their divisional train 2 or 3 miles to the north, so we knew the main body was somewhere in between.

* * * * * * *

Much of the value of aircraft will be lost if the information which they can get so quickly takes a long time to reach headquarters. If possible, machines should always land near the commander, say within a quarter of a mile, and I think the position of headquarters should be chosen with this in view. If it can not be arranged, very complete arrangements for communication with the aeroplane park must be made.

As regards the issue of orders to aircraft, I think the freer hand the commander has the better, and they should be much after the style of those issued to the commander of independent cavalry.

* * * * * * *

Then I would ask you not to spoil us. There seems rather a tendency now to make undue allowances for the Flying Corps, and anyone who thinks himself entitled to special treatment is generally a man with a grievance. I am afraid this sounds very ungrateful. There are times when we want everyone's sympathy. I am glad to say we do not look for it in vain, and in no branch of military service is sentiment of such importance as in aviation. But on ordinary occasions we ought to be judged by results, and if we

bring back no information, or, still more, false information, we deserve censure even if we have been up in bad weather. It is very pleasant to be petted when one is young, but a spoilt child is apt to develop into a very objectionable type of man, and we mean to grow up soon.

* * * * * * *

Aircraft are not going to win a campaign by themselves; they are merely an auxiliary. It is the fighting troops, the man behind the gun or rifle, the man with the bayonet or sword, that is going to win our battles. They are not going to transform a bad general into a Napoleon, an inefficient staff into a good one, nor enable infantry to march quicker.

There may be in the future rather a tendency to wait and see, to find out exactly what the enemy is doing before we act.

Cavalry will have more battle energy left and may be of greater value than ever during and after a fight.

The movements of transport may indicate the intention of the enemy to retire before the troops move.

The railheads being used for the movement of supplies may give a clue to the enemy's future movements or to a change of base.

The movements of armies of ostensibly neutral States may be discovered without incurring a *casus belli* by sending cavalry across the frontier.

To take advantage of the information available, the general must make up his mind at once, the staff must get out and distribute his orders quickly, and the troops must be prepared for longer and more rapid movements. War has not been made easier, at any rate, for the regimental soldier, as the only way in which a commander can hope to surprise his adversary is by outmarching him. There will, in fact, be a general speeding up all round, and the relative value of mobility and efficiency as compared with numbers will be greater than ever.

* * * * * * *

Ohio River Dam No. 48

BY

Maj. J. C. Oakes
Corps of Engineers; Member American Society Civil Engineers

Of the fifty-four dams proposed for the canalization of the Ohio River all of those constructed up to the present time, except No. 37 below Cincinnati and 41 at Louisville, have been constructed in the upper 300 miles of the river above the mouth of the Big Sandy. One dam, No. 29, is now under construction just above Ashland at mile 320, and contracts have been let and cofferdam constructed for locks and dams No. 31 at mile 358¼ and No. 48 at mile 804½, or 6 miles below Henderson, Ky.

All of the dams thus far built or contracted for, except No. 48, have fairly firm foundations, most of them being on rock and a few on gravel. The material at all lock sites above Louisville is supposed to be of a character that will not be easily eroded by the current, so that up to the present time no special precautions have had to be taken for protection of cofferdams during construction or for protection of the works against undermining by the current after completion and during operation.

Below Louisville, however, a totally different type of foundation is encountered. In the thirteen dams to be constructed in the lower 400 miles of the river, rock is found at the sites of only three; elsewhere foundations are fine sand and silt, so fine that the bottom changes with every stage of the river. This fact has caused considerable anxiety, not only with reference to the planning of the works to insure stability after completion, but particularly with reference to the danger to the cofferdams and erosion of the bed of the river during the period of construction. It has been openly affirmed by some of the contractors who have had experience on the Ohio River that it would be impossible to construct cofferdams in the shifting sands of the lower river that would remain during the period of construction, and, second, that if constructed, they could not be made sufficiently impervious against

seepage to withstand the ordinary pressure heads, and that they could not be pumped out sufficiently to enable the work to proceed.

Bids were opened in this office for the construction of No. 48 on September 7, 1911, and it was found that only one bidder had sufficient confidence to offer to do the work. On October 11, 1912, bids were to be opened for the construction of dam No. 43, but no bids were received. The contract for No. 48 was finally awarded to the only bidder, The Ohio River Contract Company, and during the past season the cofferdam surrounding the lock, enclosing an area of 20 acres has been constructed, pumped out, and the round piles under the river wall driven. Work was shut down for the winter on December 31, 1912, and during January, 1913, this work was submerged by one of the worst floods of record at that site, high water reaching an elevation of 371, which is within 2 feet of record high water. No particular damage has been done the coffer and it is expected that it will be in as good condition next season as is usual at other sites after the winter floods. It has therefore been proven that safe cofferdams can be constructed, maintained, and pumped out without undue trouble at the sites in question as well as in other parts of the river where better foundations exist.

It is believed that a short description of the work and type of coffer will be of interest to the member of the Corps of Engineers and also to contractors who might be desirous of bidding on future work in this section of the river.

At the site selected for dam No. 48 (fig. 1), the width between low water lines is 1,600 feet; between 20-foot contours, 3,100 feet. The lock is to be located on the convex shore of a bend in the river, the radius of which is approximately 4,000 feet. Low water is at elevation 325 (Sandy Hook datum), with high water (1884) at 373. The flood of January, 1913, at this site almost equaled that of 1884, reaching an elevation of 371.0. The lock is to be constructed on the Indiana side where the general level of the banks is at elevation 360. On the Kentucky side the bank is somewhat higher, being approximately at 380. The river wall of the lock is to be approximately along the line of low water, which places that wall some 700 feet from the contour of the bank corresponding to that of the elevation of the top of the walls. The terreplein between gate recesses is to be connected with the river bank by a causeway or dike at a general elevation of the top of the land wall.

The lock is the standard size adopted for the river, 600 by 110 feet, with lift of 9 feet. The dam is to have a navigable pass 800 feet wide with Chanoine wickets 16 feet 5 inches long; a Chanoine

weir 600 feet wide, wickets 11 feet 9 inches long, and a permanent weir 890 feet long, the elevation of whose crest is to be 1 foot below upper pool, or 337. Upper pool is at 338, lower pool 329, top of river wall 341, and top of land wall 343.

The material found at the site is a fine sand and silt intimately mixed, with an occasional pocket of very fine gravel. In driving a few long piles to a depth of 47 feet below low water some difficulty was experienced and it is supposed that a layer of gravel was encountered, but no borings have been taken to verify this. This material when quite dry or when entirely submerged in still water stands at about the slope of 2 on 3, but any movement of water either through it or over it rapidly flattens such slope to about 1 in 20. For this reason the cofferdam was kept 150 feet away from the walls.

Owing to the distance of the lock from the shore and the necessary contraction of the river, it was thought inadvisable to include any of the navigable pass in the first cofferdam, which was built therefore to inclose only the lock with arms extending to the bank. The area inclosed is about 20 acres. It is feared that this reduction of the cross section of the stream will cause considerable erosion, and it may happen that the bed of the river will be so greatly changed as to make it necessary to raise the bed before it will be possible to construct the dam as planned.

The type of coffer (fig. 2) is that known as the Ohio River box type, built 20 feet above low water and 20 feet thick for the greater part of its length; it consists of two rows of sheet piles 20 feet apart, tied together by steel rods varying in diameter from ¾-inch at the top to 1¼ inch at the bottom, held apart by separators and held together by wales on the outside 6 by 6 inches at the top, varying to 10 by 10 inches at the bottom. The space between the rows of sheeting was filled with sand removed from within the coffer by a 10-inch suction dredge, the material being excavated from the lock site. The sides of the coffer were carried into the bank, the top being of uniform elevation 20 feet above low water. To increase the stability of the coffer and decrease seepage a line of 7 by 12 inch triple-lap sheet piles, Wakefield, 26 feet long, was driven outside of and around the coffer and bolted to it.

No special difficulty occurred during the construction of the coffer and seepage through the coffer was easily controlled by three 15-inch pumps placed on a pump boat resting on piles near the front of and discharging over the coffer. Some difficulty was

met in the extension of the upper arm, owing to the fact that the river rose at a critical period and the pile driving and placing of skeleton of the coffer was carried on in water as deep as 17 feet. As this cofferdam extends some 700 feet from the bank at this stage there was, of course, a very serious current around the end, and extraordinary means had to be taken to protect the end of the coffer. Round piles with brush between them weighted down by sand bags and rock were used, and after the corner was turned a pile of riprap was placed to protect it permanently against current and ice. The banking along the front of the coffer was protected by several masses of rock (fig. 4) forming short spur dikes to prevent a racing current alongside the dike and the whole length of banking was covered with gravel. (Fig. 2.) This gravel was fine, the largest particles being not more than one-half to three-fourths of an inch in diameter, and some of the material was practically nothing but coarse sand. In my opinion, this gravel is much too fine for the purpose intended and the contractor was so notified. He feels, however, confident that it will serve its purpose, and it will be very interesting to see whether his belief is justified.

The greatest trouble met in the unwatering of the coffer was in protecting the banking against the inside of the coffer from undermining by seepage and drainage water. Wherever there was flowing water the sand was eroded and the banking gradually sloped until it was almost flat. To prevent this, sandbags were used freely, and practically the whole caving surface was covered with the same kind of gravel as that used on the outside. For this protection this gravel was very suitable, and as soon as the sand was covered by a thin layer further erosion did not take place, and the banks remained with a slope of about 2 on 3.

It is believed that the Wakefield sheet piling driven around the coffer were not used to the best advantage. These piles should have been driven deeper, so as not to overlap the coffer sheeting over 5 feet, instead of about 13 feet as actually driven, and should have been driven close to the coffer. The driving of a great part of the Wakefield piles occurred at medium stages of the river (10 to 17 feet) with the result that the driving was poorly done and this sheeting is not as tight as it should have been. The contractor was unable to find men who had adequate experience in driving piles in sand with the use of a jet. Both in the case of the sheet piles and the round piles under the river wall the pile drivers did not accomplish more than 50 per cent of what they should have

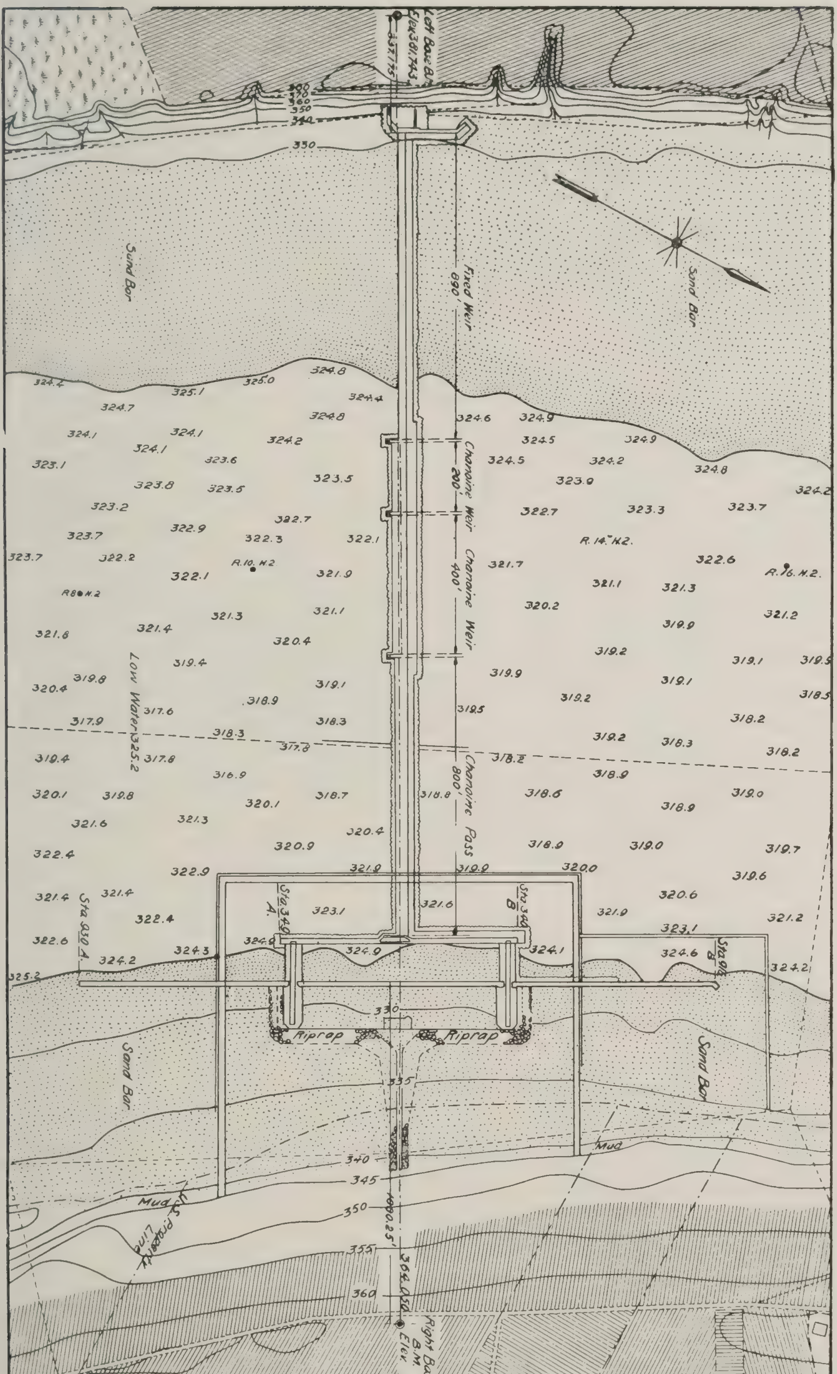


Fig. 1. Site of Dam No. 48, Ohio River.

accomplished. This was due in part to lack of experience, but also in part to improper pumps, hose, and jets. Up to the end of the season the greatest number of round piles, 30 feet long, driven by one pile driver in one day was 31. While the jet was used it is very doubtful whether any benefit was obtained, owing to lack of pressure at the nozzle.

The contractor's attention has been called to these defects, and preparations are being made to provide the pile-drivers with proper jet apparatus, and it is believed that when he begins to drive next season each pile driver will drive from 80 to 100 piles per day.

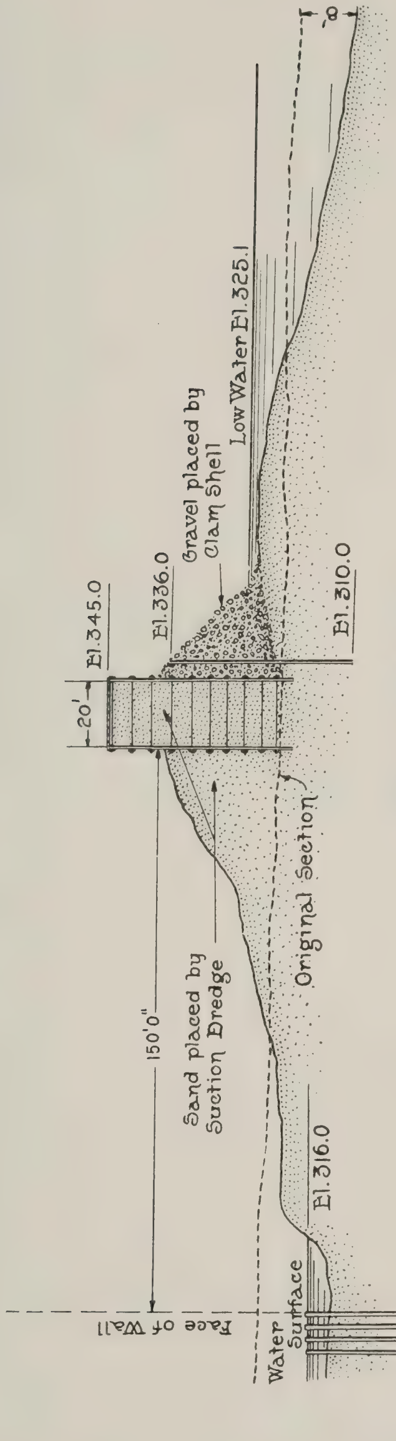
The use of a pump boat with all of the pumps concentrated, the boat resting on piles as the water lowers, has been excellent and it is believed is very economical and much preferable to other methods commonly used. When the coffer is flooded the boats are simply disconnected from the discharge pipes, their suctions raised, and they are floated out of the coffer through the passway. This furnishes a very simple and economical manner of removing and placing the pumping apparatus.

This being the first coffer to be placed on sand foundations in the Ohio River, the contractors have taken very great care to use the best material and to make everything as secure as possible. It is possible that in future cofferdams some of the measures adopted in this case may be found to be unnecessary and the cost of the cofferdam material reduced.

The following is an extract of a report of the Government inspector in charge of the work, Junior Engineer Edward H. West, in which the work is described in fuller detail:

* * * * * * *

The coffer, both inside and out, is heavily banked with sand. A typical cross section is shown on fig. 2. Plans were closely followed in construction, the only exceptions being as follows: For a distance of 670 feet from the land end of the upstream wing and 520 feet from the land end of the downstream wing, tie-rods were spaced 8 feet on centers instead of 6 feet on centers; in some portions of the coffer, instead of 2 by 12 inch deck joists spaced 24 inches on centers, 2 by 10 inch joists were used, spaced 16½ inches on centers, and in a short portion of the river arm 2 by 10 inch joists 18 inches on centers were used. The sheet piles were driven from a floating pile driver, and at times the river was too high to permit the piles to be driven to the proper depth; consequently, such piles do not have quite the penetration intended, the maximum



OHIO RIVER
LOCK AND DAM No. 48
Typical Section of Completed Cofferdam
Scale of Feet
Vert. 0 5 10 20 30
Hor. 0 10 20 40 60

Fig. 2.

difference between actual and intended penetration being about 3 feet; at the upstream outer corner the length of a number of sheet piles was increased to 32 feet for additional safety at the point of supposed greatest weakness; at this corner about fifty round piles were driven as shown on fig. 4, the area inclosed within them was filled with brush weighted with sandbags. Afterwards the entire corner was protected with derrick-stone piled as high as the top of the coffer. The lower outer corner was also protected by derrick stone and several stone dikes were built along the river side, as shown on fig. 1.

On June 20, 1912, the line of sheet piles was commenced, beginning at a point 250 feet from the land end of the upper wing, and piles were driven on fifty-six days, an average of 40 piles per day. On a number of these days, however, two pile drivers worked, the average number of piles per day for one driver being not more than 30. These sheet piles were driven rather carelessly, making alignment poor, and leaving the joints not very tight. Better progress could have been made in driving them, but as it was only necessary to keep ahead of the coffer skeleton no great effort was made to push the pile driving. After the driving of sheet piles had been begun, trenches about 2 feet deep and 20 feet apart were dug parallel to the sheet piles. In these a skeleton was erected consisting of wales and those pieces of sheeting through which the tie-rods passed, the sheeting being driven about 2 feet into the sand. All wales were scarfed for 2 feet at both ends and holes bored for tie-rods through the center of the scarf. Where the spacing of tie-rods was 8 feet 18-foot timbers were used for wales and where the spacing was 6 feet 20-foot wales were used, thus allowing a lap of 2 feet at each end. At each tie-rod a temporary separator perpendicular to the wales and 20 feet long was placed and the nuts on the rod tightened. The remainder of the sheeting was then driven and after the proper cut-off elevation had been marked the ribbing strips were spiked on and the sheeting cut off to grade. Cracks between adjacent pieces of sheeting were closed by 1 by 3 inch battens nailed on the inside. When the coffer had been extended into water so deep as to prevent further work from land the skeleton was bolted together on a small barge, one section at a time, and as each section was completed it was lifted by a derrick boat until the barge could be moved forward, when it was lowered into the water (fig. 3). Sheeting was then driven by men standing on the wales. Throughout the construction two gangs of

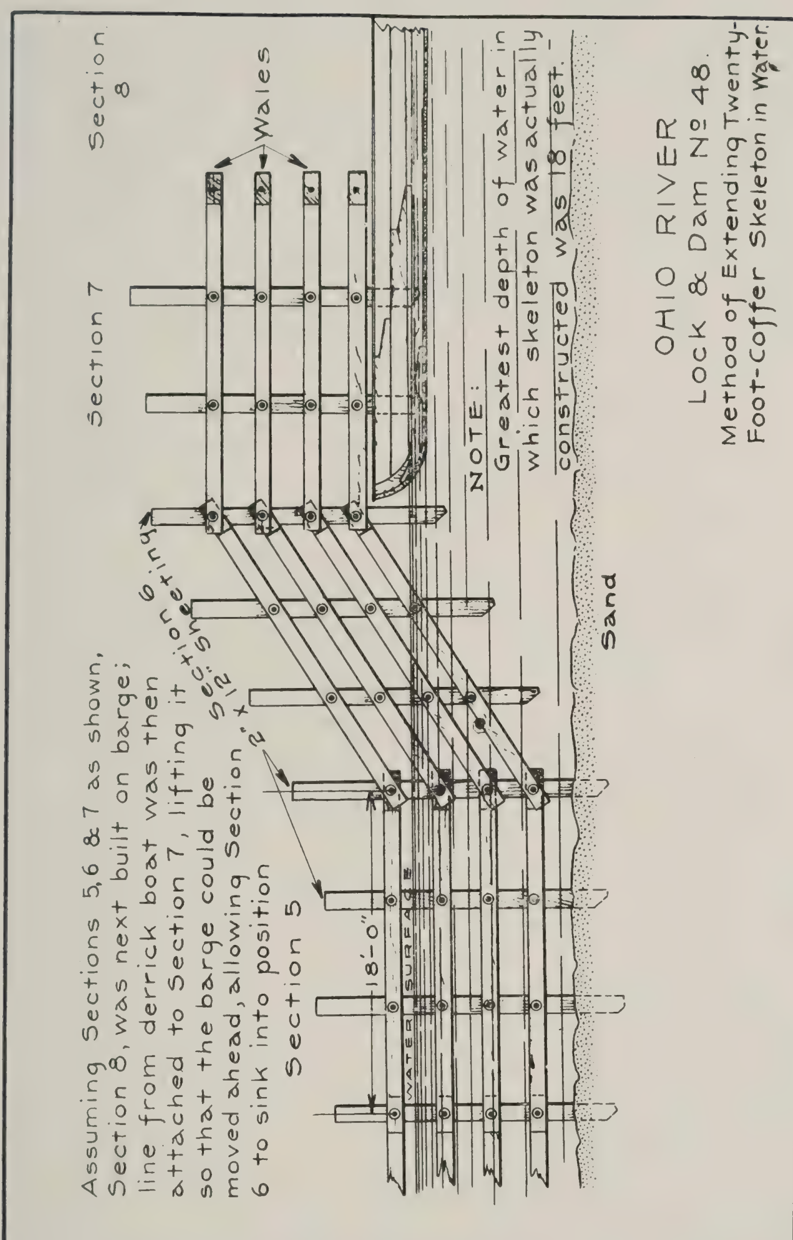

Assuming Sections 5,6 & 7 as shown, Section 8, was next built on barge; line from derrick boat was then attached to Section 7, lifting it so that the barge could be moved ahead, allowing Section 6 to sink into position
Section 5
Section 6
Section 7
Section 8
Wales
2" x 12" Sheeting
18'-0"
WATER SURFACE
NOTE: Greatest depth of water in which skeleton was actually constructed was 18 feet.
Sand
OHIO RIVER
Lock & Dam No 48.
Method of Extending Twenty-Foot-Coffer Skeleton in Water.

Fig. 3.

carpenters worked; at times each gang extended both skeleton and sheeting, but better progress was made when one gang worked on skeleton only and the other followed up with sheeting. Each gang contained at different times from ten to twenty men; it was found that not more than twenty men could work together to advantage. In general, it may be said that the rate of progress depended upon the rapidity with which the skeleton was advanced, no difficulty being experienced in keeping close up with the other work. As a matter of fact, sheeting was frequently delayed in order that the skeleton might be extended. During the season the skeleton was actually extended on about seventy-one days an average of 38 feet feet per day; this value corresponds to two wale lengths, either 36 or 40 feet, depending on the length of the wales being used; it was noted in the field that two wale lengths constituted an average day's work. Sheeting was actually driven on fifty-three days, an average of 52 feet per day (double row) from a minimum of 10 feet to a maximum of 103 feet, but on a great many days on which sheeting was driven much other work was done by the same gang and the above values are not reliable for progress estimates. It is believed that one gang can extend a double row of sheeting 100 feet per day. The extension of skeleton may be assumed to be work accomplished by a single shift since night work on this portion of the coffer is negligible; sheeting, however, was driven quite satisfactorily at night. After the sheeting had been cut off to proper elevation, the deck joists were laid and spiked to the ribbing strips. This having been done, the discharge pipe of the 10-inch suction dredge was moved into position and the sheeted portion of the coffer filled with sand taken, if possible, from the "pay excavation" for the lock walls. Bulkheads across the coffer were built at convenient intervals as the construction advanced. As the coffer filled, the separators were removed and afterwards used again. At first, a long trough built on top of the deck joists of rough lumber, on a grade of 1 per cent, was used for distributing the sand, but it proved to be very inefficient and was discarded; subsequently the desired distribution was accomplished by moving the end of the discharge pipe. Short time tests of the suction dredge showed that it could place 100 cubic yards of fill per hour, through 300 feet of discharge pipe, maximum lift 20 feet. It handled a total of about 100,000 cubic yards of material. The coffer contains only about 40,500 cubic yards; the remainder was

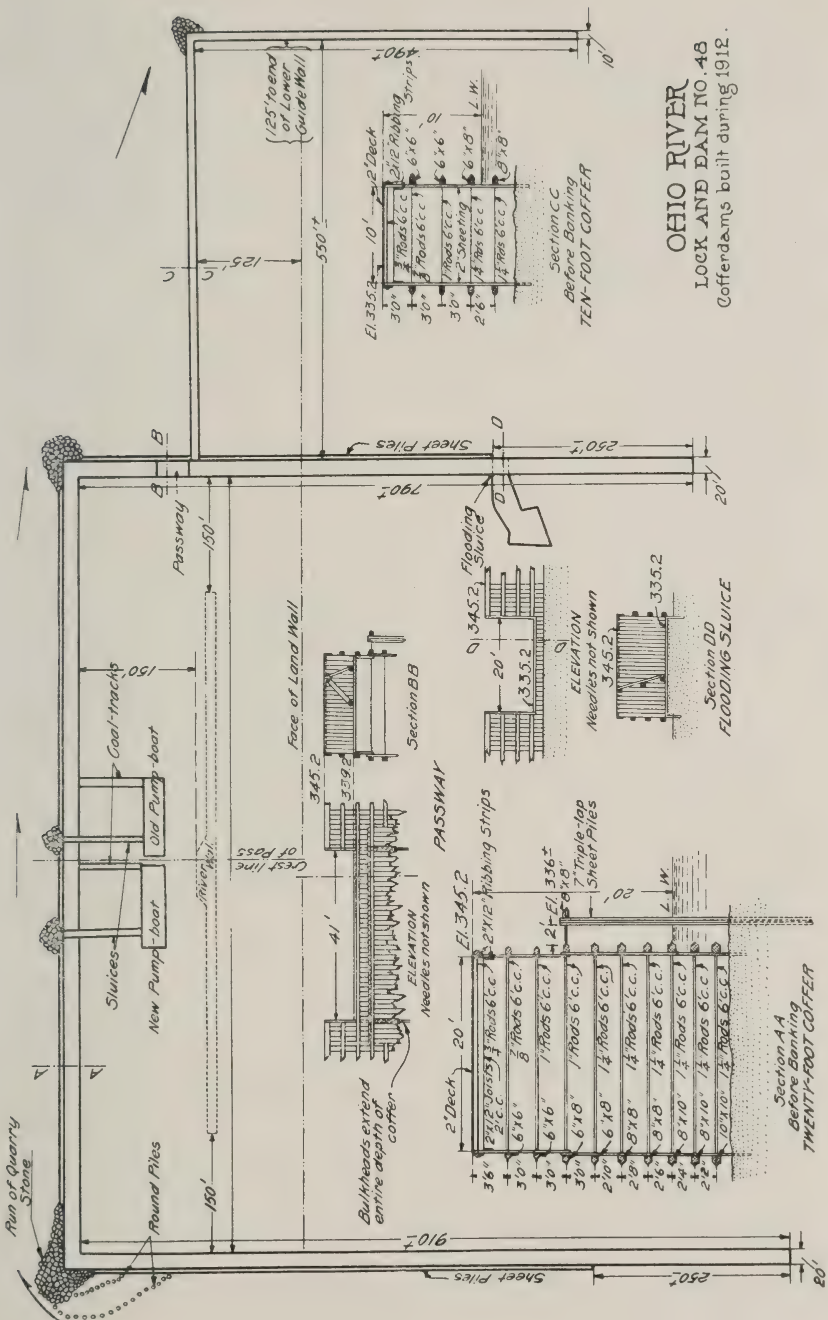
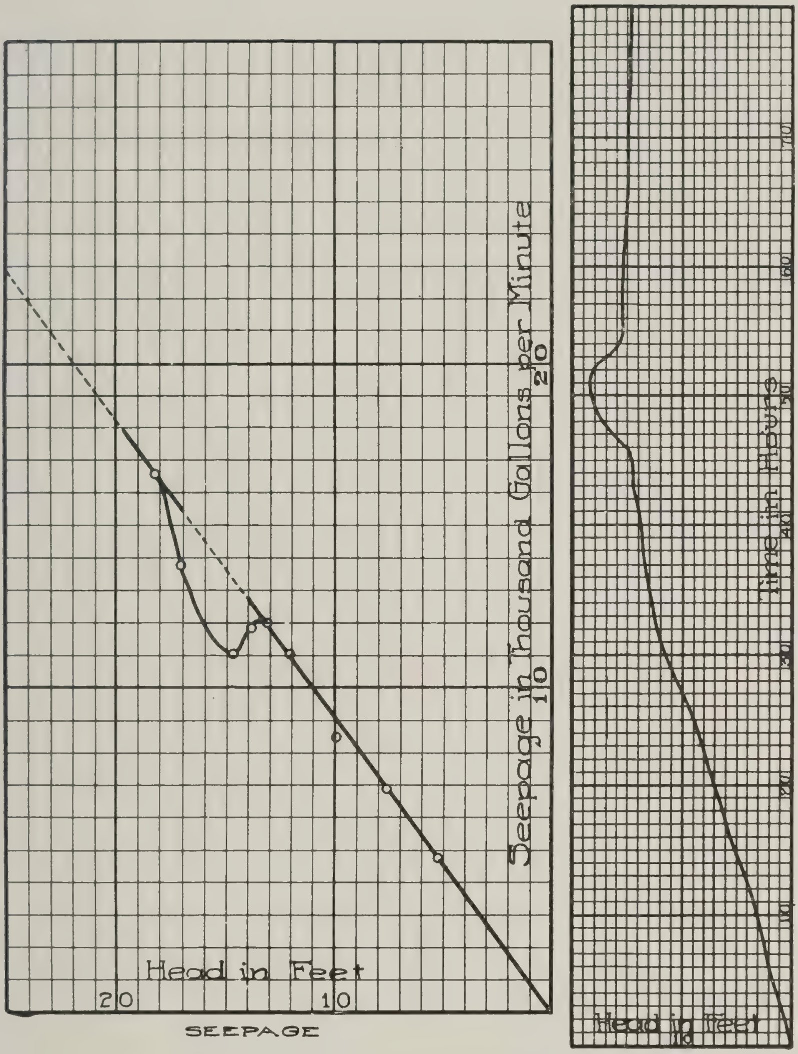

OHIO RIVER
LOCK AND DAM NO. 48
Cofferdams built during 1912.
Section CC
Before Banking
TEN-FOOT COFFER
Section BB
PASSWAY
ELEVATION
Needles not shown
Section DD
FLOODING SLUICE
Section AA
Before Banking
TWENTY-FOOT COFFER
Face of Land Wall
Sheet Piles
Round Piles
Run of Quarry Stone
Sluices
Coal-tracks
Old Pump-boat
New Pump-boat
Passway
Flooding Sluice
Crest line of Pass
125' to end of Lower Guide Wall
Bulkheads extend entire depth of coffer
7" Triple-lap Sheet Piles

Fig. 4.

used for banking. Of the total excavation made, about 50 per cent was within the specified limits of "pay excavation." As shown on fig. 4, a flooding sluice was built in the lower wing and a passway for removing boats, etc., was also left in the lower wing. Discharge sluices and coal tracks were constructed in the river side near the crest line of the dam. A layer of fine gravel was spread over the whole river side of the coffer-banking to prevent scour as much as possible. It is believed that the one flooding sluice will be all that is necessary, unless very extraordinary conditions arise; in flooding the coffer when the river is near its top and too great a volume of water entering would endanger the permanent work, it is intended to place blocks behind alternate needles, as is often done in regulating pools above needle dams. In the event that this scheme should fail to pass a sufficient quantity of water to fill the coffer before it is topped by a rise, it is intended to use the passway as an emergency opening after constructing a temporary sluiceway.

The tongue pieces for sheet piles were dressed in the contractor's planing mill, as was all material requiring finish, such as sheeting for sluiceways, needles for closing sluices and passway, etc. Two pile drivers were used during the season, both mounted on decked barges, with cabins. One barge was 20 by 50 by 4 feet (No. 30) drawing 25 inches, and the other (No. 29) 22 by 60 by 5 feet, drawing 25 inches. On No. 30 was one 40-horsepower locomotive firebox portable boiler, made by the Brownell Company, Dayton, Ohio; one two-drum hoisting engine with four winch-heads, made by the American Hoist and Derrick Company, cylinder diameter 7 inches, length of stroke 10 inches, rated horsepower, 20; the hammer was a No. 2 steam hammer, made by the Vulcan Iron Works, Chicago, weight of moving parts 3,000 pounds, gross weight 6,500 pounds, strokes per minute 60; the make or size of the pump is not known, but its indicated pressure varied from 80 to 110 pounds, the pressure at the jet is unknown, but pressure loss between pump and jet was considerable; the jet was a 2-inch pipe reduced to 1¼ inches at the nozzle. On No. 29 was one 60-horsepower locomotive firebox portable boiler, made by the Brownell Company; one nondescript engine on Lidgerwood base with two winch heads, cylinder diameter 6¼ inches; length of stroke, 8 inches; horsepower, 12. The hammer was a No. 3 steam hammer made by the Vulcan Iron Works; weight of moving parts, 1,800 pounds; gross weight, 3,800 pounds; number of strokes per minute, 60; a McGowan pump



OHIO RIVER
Lock and Dam No. 48.
Seepage through 20' Lock Coffer

Fig. 5.

supplied water to the jet at about 90 pounds pressure, as indicated at pump, but it is believed that the greater part of this was lost before reaching the nozzle; the jet was a 2-inch pipe, reduced at nozzle to 1¼ inches. At different times, three derrick boats were used; two of these were identical, having been recently built for this contract; the third, No. 31, was an old boat, quite inefficient and nearly worn out. Nos. 33 and 34 were built on hulls 34 by 66 by 4 feet 9 inches, drawing about 20 inches; the cabins are 28 by 28 feet. The one boiler on each boat is a locomotive firebox portable boiler, 40 horsepower, made by the Houston-Stanwood & Gamble Company, of Cincinnati; each engine a 50-horsepower three-drum tandem engine for derrick boats, built by the Lidgerwood Manufacturing Company, cylinder diameter, 10 inches: length of stroke, 12 inches; drums, 16 by 24 inches; the swinging engine is a 6-horsepower Lidgerwood, cylinder diameter 5 inches, length of stroke 6 inches. The fittings were made by the American Hoist and Derrick Company. The boom is 65 feet long and the mast 36 feet. Each boat has three spuds. All excavation, banking, and filling was done by a suction dredge; hull, 22 by 100 by 5 feet 6 inches, drawing 24 inches, with one spud at the suction end. The two boilers are each 60-horsepower, built by the Houston-Stanwood & Gamble Company; one engine, 100 horsepower, built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches. The pump is a 10-inch centrifugal sand pump, belt driven, made by the Morris Machine Works, Baldwinsville, N. Y., with 10-inch suction and 12-inch discharge pipes. For pumping out the coffer two pump boats were available; one, built especially for the purpose, contains, on a hull 36 by 106 by 5 feet 6 inches, three 100-horsepower boilers, made by the Houston-Stanwood & Gamble Company; three 100-horsepower engines built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches; and three 15-inch centrifugal pumps, belt driven, made by the Morris Machine Works, and having 18-inch suction and 15-inch discharge pipes. The boilers consumed 330 bushels of coal per 24 hours during a run of 18 days, and each pump, discharging 26 feet above water surface, handled 8,000 gallons per minute. The old pump boat has never been used on this contract; it contains two Brownell boilers, 60-horsepower each; one engine, made by Charles Barnes & Co., Cincinnati, cylinder diameter, 15 inches; length of stroke, 18 inches; one engine, built by the Nagle Engine and Boiler Works, Erie, Pa., cylinder diameter, 12 inches; length

Fig. 6. Cofferdam after closing, showing character of soil, round-piles under river-wall, pumpboats, etc.

of stroke, 16 inches. Both centrifugal pumps were built by the Morris Machine Works, Baldwinsville, N. Y.; one is a 15-inch pump and the other an 8-inch sand pump, both belt driven. The floating machine shop contains a lathe made by the Hamilton Machine Tool Co., Hamilton, Ohio, a pipe-threading machine made by the Merrell Manufacturing Company, Toledo, Ohio, a pipe-threading machine and a drill-press made by Davis & Egan, Cincinnati, Ohio, and a hand-made forge.

The cofferdam was closed November 12, 1912, and a typical cross section of river side is shown on Fig. 4. Inside were left both pump boats; the old derrick boat for use as a clam-shell dredge; and both pile drivers. A temporary sluiceway was constructed, discharging through the passway, and a coal track was built to supply fuel to the new pump boat *Uncle Joe*, which was temporarily located over the hole excavated for the lower gate track and recess. It was intended to pump down low enough to enable the pile drivers to drive 126 piles for supporting the pump boats in their permanent locations. Pumping was begun on Friday, November 15, 1912, at 2 p. m. and was continued until Saturday, November 16, 1912, at 3.10 p. m., when the mouth of the discharge sluice was covered by the rising river and it became necessary to stop the pumps. On Thursday, November 21, the river had fallen sufficiently to begin pumping and the pumps were started at 9.15 a. m. and ran continuously until the piles for pump boat foundations were driven and capped. Gages had been set inside and outside the coffer and an inspector was kept on the pump boat continuously during the first pumping. The slope of the discharge water was measured in the sluiceway by a series of small gages set before pumping commenced, and the actual discharge while the pumps were making 350 revolutions per minute was computed by Kutter's formula, using a coefficient of rugosity of .009, and found to be 24,000 gallons per minute for three pumps. An attempt was made to obtain more or less accurate estimates of the seepage through the cofferdam at different heads. Since the pumpboat was floating the vertical distance from water surface to center line of discharge pipes did not vary and the discharge was assumed as constant at 480,000 gallons per pump-hour as measured. The inside gauge was read as closely as was possible (to hundredths of a foot) and recorded hourly and a careful record kept of the actual number of pump-hours run. The outside gauge was read at 8 a. m., 12 m., and 4 p. m., and the river stage at other hours could readily

be estimated. From the readings of these two gages the head on the coffer at any time could be computed. A careful survey had been made of the interior of the coffer by soundings on October 31; from the data obtained by this survey a contour map was plotted on a scale of 1 inch equal 40 feet, contour interval 1 foot, and areas were measured by planimeter at each foot of elevation through the probable range of water surface during pumping. The area of water surface at each hour was obtained by interpolation and the amount by which the water content of the coffer was reduced was computed by average end areas; the seepage during each hour was taken as the difference between the amount of water handled by the pumps and the amount by which the coffer content was reduced. For example:

Date: November 22, 1912.

Time p. m.	Outside Gage.	Inside Gage	Head.	Pump hours run.	Water actually pumped.	Content reduced.	Seepage.	Seepage per minute.
7	332.53	319.79	12.7					
				2.00	960,000	144,610	815,390	13,589
8	332.50	319.68	12.8					
				2.00	960,000	259,870	700,130	11,669*
9	332.48	319.48	13.0					
				2.00	960,000	230,510	729,490	12,158†
10	332.45	319.30	13.2					
				2.00	960,000	252,380	707,620	11,794
11	332.43	319.10	13.3					

*Average Head=13.0.
†Seepage per Minute=12,052.

It was impossible to read the inside gage closely enough to make the seepage estimates for each hour agree perfectly, and so in plotting the curve shown (on bottom, fig. 5) the points plotted are average values for several successive hours. While conditions were such that an absolutely accurate estimate of seepage is impossible, the values as given are believed to be close enough for all practical purposes. It would seem that the seepage should vary as the square root of the head, following the well-known laws for flow through pipes and orifices, the passage of the water through sand amounting to flow through an infinite number of minute pipes, but the values actually obtained as outlined above indicate a straight line variation, that is, directly as the head (top of fig. 5).

It will be noticed that from a head of 13 feet to 15 feet the seepage appears to rapidly decrease and from 15 feet to 18 feet it appears to increase more rapidly than normal. This phenomenon

occurred during the night when no attempt was being made to lower the water surface rapidly. One pump had been cut out and the other two speeded up so that they were pumping more than the quantity of water measured at normal speed, but how much more is not known; consequently, the seepage appears to decrease, but in reality such is not the case. After the piles for the pump boat foundations had been driven and capped the coffer was allowed to fill, the boats were floated into position and the *Uncle Joe* pumped down, allowing them to settle on their foundations. Pumping was continued until the water surface inside reached an elevation of 310.7, when the coffer was accepted as satisfactory; it was then allowed to fill up to 316.0, and driving of round piles for river wall foundation was begun from floating drivers.

Seepage was apparently uniform around the entire coffer perimeter, since a single large leak would have resulted in great damage and the wrecking of a portion of the cofferdam. Naturally, this seepage water collected into a relatively small number of rather large streams, moving toward the pumps, each of which carried a large quantity of sand, which was deposited in the still water where the pile drivers were floating. Taking care of the seepage was a simple matter, but serious difficulty was encountered in preventing the excavation from filling with this sand. The clam-shell dredge proved to be an inefficient and costly method of solving the problem.

On several occasions pile driving was suspended until sand thus washed in could be removed. It was thought that by building dams of sand bags across those streams the movement of sand could be stopped. This was done and proved to be very advantageous, but when water began to flow over the dams sand continued to move into the excavation, though in diminished quantities. A further diminution in this sand movement was effected by spreading a 6-inch layer of gravel over the greater part of the river side of the interior banking. In spite of all efforts, however, large quantities of sand continued to move into the ''hole'' and at the time work was suspended for the winter the problem of driving piles without constant excavating was still unsolved.

* * * * * * *

Break in the Illinois and Mississippi Canal at Aqueduct 4

BY

Maj. CHARLES KELLER
Corps of Engineers; Member American Society Civil Engineers

Aqueduct 4 of the Illinois and Mississippi Canal is about 1¾ miles west of Lock 22, the latter being at the west end of the summit level of the canal. The level between Lock 22 and Lock 23, which, besides Aqueduct 4, contains also Aqueduct 5, is about 9 miles long. The railroad station nearest to Aqueduct 4 is about ¾ mile away at Mineral on the main line of the Chicago, Rock Island and Pacific Railway.

On August 14, 1912, a break occurred at Aqueduct 4, the nature of which and whose repair are described in the following report of Assistant Engineer C. M. Waters:

On the morning of August 14, 1912, when the patrolmen and laborers on Overseer Crist's section were on their way to work, they found a leak at the south end of the west abutment of Aqueduct 4, and notified Overseer Crist (by telephone from a neighboring farmhouse) who immediately notified me. Instructions were given the overseer to rush his men to the scene, close the spillway at Lock 22 and begin feeding heavily through Lock 23, and, if possible, close the leak with sacks. About an hour later, when I arrived at the aqueduct, the water appeared to be coming under the southwest wing wall close to the canal prism. Water in the creek had then raised to almost the height in the canal, so it was difficult to tell just where the leak was. A few minutes after I arrived, the south and west wings of the south end of the abutment settled down slowly, the bank settling with them. The water then soon overtopped the bank. The creek, which is formed by Coal and King creeks uniting just above the aqueduct, is a drainage district, as are also the creeks forming it. Levees have been built for a long distance along the creeks above and below the aqueduct. The levees prevented the water from spreading out over the farm lands, and also prevented the water from being drawn out of the canal through the break at a very rapid rate.

In building the abutment the contractors divided it into three

sections, one of these divisions coming in line with the north side of the west wing of the abutment, shown on the drawing as C-D. All that portion of the abutment south of this line was wrecked, together with all but two 5-foot sections of the southwest slope paving. One of these sections was later broken up to relieve the men from the danger of its falling upon them.

The entire wrecked portion of the abutment was crowded to the east, at the base, about 9 feet. The south and west wings separated at the angle joining them, and also broke through several horizontal cracks. The south wall was left inclined to the west on a $\frac{1}{2}$:1 slope, and to the south on a 4:1 slope. There was no movement to the south of either wing. The west wing settled down close to the remaining portion of the abutment, dipping to the west at an angle of about 30 degrees. The wrecked slope paving was piled in the angle between the wing walls.

A deep hole was washed under the south end of the breast wall, exposing the foundation piling for a distance 8 feet north from section C-D. The breast wall and reinforced concrete lining were uninjured.

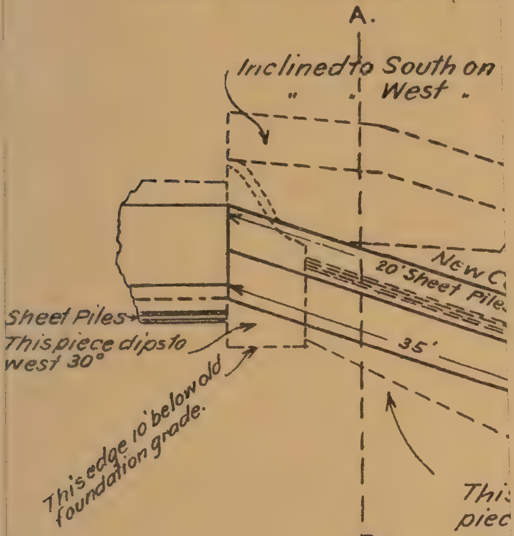
In making the repairs, earthen cofferdams were thrown across from the west bank of the creek to the west pier, above and below the aqueduct, and the inclosed area pumped out. The movable dam in the west end of the aqueduct was raised, holding 3.8 feet of water east thereof. A cofferdam was built across the canal prism west of the break. A wooden flume was built to carry the water from the aqueduct past the cofferdam in the canal prism. This flume served the double purpose of helping to maintain the lower levels and of taking care of the leakage through the valves at Lock 22.

The top of the south wing wall was removed to about 10 feet above the foundation, and the west end of the west wing wall was removed down to about the grade of the original foundation. It was easier to do this than to move the pile-driver around it.

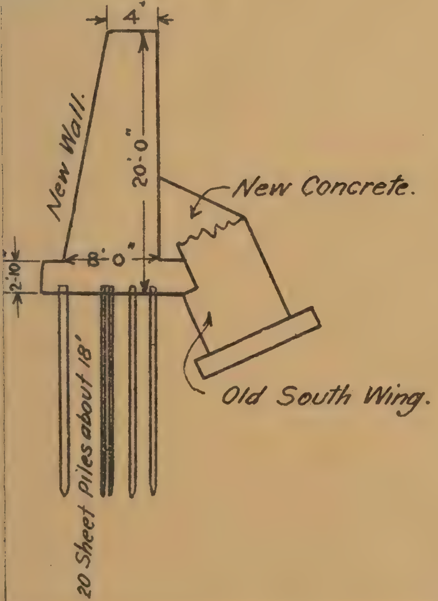
Triple lap sheet piles of yellow pine, dressed to 2$\frac{5}{8}$ by 11$\frac{1}{2}$ inches and 20 feet long, shod with cast iron, were driven in a row about 30 feet long southwest from the south side of the old west wing, over which was built a concrete foundation 2 feet 10 inches deep, for the new wing wall which is 37 feet long on the east side, 35 feet on the west, 8 feet at the base, and 4 feet at the top. The foundation of this wall was made to extend east to the wrecked south wing, and later the area between the new wall and the old was filled with concrete to the top of the old wall as it was left. Thirty-two 20-foot piles were driven in this foundation.

After this wall was completed, a row of thirty of the same kind of sheet piles was driven southwest from the outer end of the concrete. These were driven to a good resistance, which left the tops at about bottom grade at the north end and a few feet higher at the south end. A similar row of sheet piling was driven from the north side of the west wing across the canal prism about 18

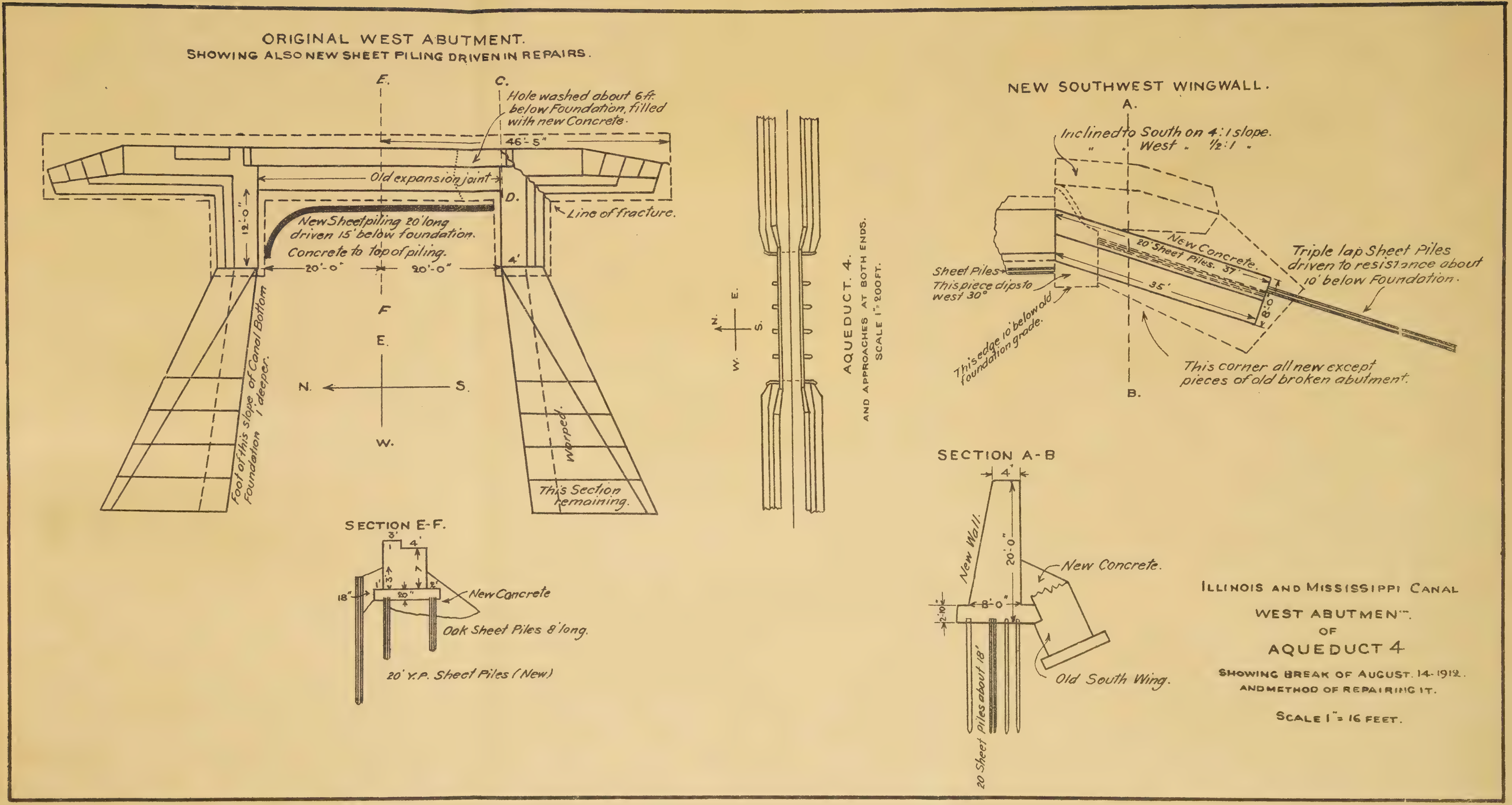
NEW SOUTHWEST WIN



SECTION A-B



IL


ORIGINAL WEST ABUTMENT.
SHOWING ALSO NEW SHEET PILING DRIVEN IN REPAIRS.
Hole washed about 6ft. below Foundation, filled with new Concrete.
46'-5"
Old expansion joint
Line of fracture.
New Sheetpiling 20' long driven 15' below foundation.
Concrete to top of piling.
12'-0"
20'-0"
20'-0"
4'
Foot of this slope of Canal Bottom Foundation 1' deeper.
Warped.
This Section remaining.
N.
S.
E.
W.
SECTION E-F.
New Concrete
Oak Sheet Piles 8' long.
20' Y.P. Sheet Piles (New)
AQUEDUCT. 4.
AND APPROACHES AT BOTH ENDS.
SCALE 1" = 200 FT.
NEW SOUTHWEST WINGWALL.
Inclined to South on 4:1 slope.
" " West " 1/2:1 "
New Concrete.
20' Sheet Piles. 37'
35'
Sheet Piles
This piece dips to west 30°
This edge 10' below old foundation grade.
Triple lap Sheet Piles driven to resistance about 10' below Foundation.
This corner all new except pieces of old broken abutment.
SECTION A-B
New Wall.
20'-0"
8'-0"
New Concrete.
Old South Wing.
20 Sheet Piles about 18'
ILLINOIS AND MISSISSIPPI CANAL
WEST ABUTMENT
OF
AQUEDUCT 4
SHOWING BREAK OF AUGUST. 14-1912.
AND METHOD OF REPAIRING IT.
SCALE 1" = 16 FEET.

inches west of the breast wall and curved westward to the west end of the west wing on the north end of the abutment. The space between this row and the breast wall was filled with concrete to the top of the sheet piles, which were 2 to 3 feet above the foundation. The deep hole under the breast wall was well cleaned out and filled with concrete. Care was taken in doing this to crowd the concrete to the north under the original foundation. Along the east side of the breast wall a trench was excavated about 6 feet deep and filled with concrete. Holes were drilled through the foundation at the east side of the breast wall and grout poured in to fill any openings under the foundation.

The west wing wall was not replaced. The longer new wall made it possible to set the bank back to the south, so that the west wing wall and vertical slope paving could be dispensed with. The new bank was riprapped after water was turned in. A study of the wreck shows that the west end of the west wing lies the deepest, which leads me to believe that the leak started around the west end of that wall under the slope paving, followed back of that wing and found its way out under the north end of the south wing. The water was turned past the break about noon September 14, and a boat locked through the same afternoon. (See Plate.)

Following is a cost statement:

Cement	$431.51
Lumber	736.73
Coal	6.00
Gasoline	6.10
Kerosene	6.00
Oils	1.00
Piles, round	100.00
Repairs	90.52
Livery	19.50
Travel	20.00
Subsistence	31.89
Cast iron pile shoes	99.88
Hardware	4.05
Gravel	274.56
Freight	22.80
Bolts	7.29
Rope	10.95
Labor	4,640.69
Total	$6,509.47

Lock 22 is farther from the railroad than is Aqueduct 4, so that such materials as came by rail were hauled direct to the aqueduct from Mineral. These included piles, timber, bolts, etc. Gravel, sand, cement, and mechanical equipment were procured from the eastern section of the canal, carried by barge as far as Lock 22, and from there to the aqueduct by wagon. No lockages were made

through Lock 22, as to do so would have made more difficult the problem of handling the water at the aqueduct.

The remains of the two concrete wing walls were drilled by hand and broken up with powder. The masses were so small and so peculiar in form as not to justify the use of mechanical drills. Piles were driven by steam driver in the pit and concrete mixed by an engine driver mixer on the northwest approach to the aqueduct.

Mr. Waters' theory as to the manner in which the wings of the aqueduct were undermined seems correct. The plans of this aqueduct, herewith, show there were two rows of triple-lap piles under the breast wall, one row extending to the ends of the transverse wings, the other turning at right angles and extending under the longitudinal wings. These sheet piles extended 8 feet below the lowest concrete and, as the canal bed was at level with the top of the breast wall of the abutment, it follows that, if the sheet piles were tight as, from the circumstances of the case seems reasonably certain, to undermine the foundations of the wing walls it was necessary for a considerable stream of water to create a channel for itself through a vertical thickness of somewhere near 17 feet of earth. Until the round piles had been deprived of much of their supporting power, the abutments could not fail. The round piles were 16 feet long and, presumably, would not have failed until by the washing out of the sheet piles a channel had been created in which scour of the earth remaining about the round piles might readily occur. Assistant Engineer Waters has stated that in his opinion the round piles under the west wing settled, or were forced down, vertically, while those under the south wing were carried over with it when it fell, so that their tops were deflected to the east. The bed of the canal at this locality is firm and quite impervious, so that the formation of a leak sufficient to cause destruction of the wings in the manner described is considered to be noteworthy.

Gouverneur Kemble Warren*

BY

Brig. Gen. HENRY L. ABBOT
Corps of Engineers, Retired

It would be useless to attempt within our narrow limits to review the life work of a man so distinguished in many fields of honorable ambition as was General Warren. His scientific record will find a place in the memoirs of the National Academy of Sciences, of which he was long a member. It is peculiarly for us, sons of a common alma mater, to cherish the memory of his soldierly achievements, which have reflected honor upon ourselves and upon our profession.

Gouverneur Kemble Warren was born on January 8, 1830, at the village of Cold Spring, within hearing of the morning and evening gun at West Point. He was the fourth in a family of twelve children—eight sons and four daughters. As a boy he was educated at the schools of his native place, and for one year at Kinsley's Classical and Mathematical School near West Point, where he was a student when his cadet appointment was received. He entered the Military Academy on July 1, 1846, at the early age of sixteen, and was graduated on July 1, 1850, standing second in a class of forty-four members. He was at once assigned to the Corps of Topographical Engineers, in the grade of Brevet Second Lieutenant.

The first duty which devolves upon a young officer often exerts an enduring influence upon his professional character; and Lieutenant Warren was fortunate in the experience which he gained as assistant to Captain (now General) Humphreys upon the investigations and surveys of the Mississippi delta. The work was onerous, and peculiar circumstances threw him into more than usually intimate relations with his chief, for whom he formed a strong personal attachment which lasted through life.

Lieutenant Warren's first opportunity for original research occurred in 1854, when he was assigned to the duty of compiling a general map of the region west of the Mississippi. The country

*Reprinted from the Proceedings of the Association of the Graduates of the U. S. Military Academy, Annual Reunion of June 12, 1883.

was then a wilderness intersected by a few lines of reconnaissance, and the work demanded laborious and judicious analysis. The resulting map and memoir, dated 1858, exhausts all valuable material from the earliest discoveries to its date, and will remain a standard historical authority. This work was performed under the pressure of other duties and largely at night. During its progress he devoted much labor to the joint report (1854) of Captain Humphreys and himself upon Pacific Railroad explorations, and also conducted three separate explorations in Dakota and Nebraska.

The first of these explorations was made as the Engineer officer of General Harney's staff, in his campaign against the hostile Sioux, memorable for the victory of Blue Water Creek on September 3, 1855. One little incident connected with this expedition illustrates Warren's character. He had been sent up the Missouri to Fort Pierre on duty, while the column was forming at Fort Kearny. Time was lacking to rejoin General Harney by water before the march began. The direct overland route (300 miles) led through the heart of the enemy's country and was wholly untravelled and unknown. Against the earnest advice of his brother officers at Fort Pierre, including the commanding officer, who regarded his destruction as certain, Warren organized a little band of seven half-breeds and prairie men, successfully made the march in two weeks, and mapped his route. This exploit, apparently so rash, was in truth the result of an intelligent study of the chances. The weather was yet too warm for the probable formation of roaming war parties, especially as it was the season for making "sweet corn." By using no tents or fires at night, and by marching under cover of darkness when near an enemy, Warren reasoned that the well armed and alert little band could run the gauntlet—and he was right. Throughout his life he never lacked sagacity to plan or courage to execute.

Lieutenant Warren's explorations of 1856 and 1857, covering many hundred miles, were made with small parties among powerful and semi-hostile tribes, for the purpose of obtaining the information necessary for subduing them and for opening the country to civilization. He was the first explorer of the now celebrated Black Hills, passing through their eastern, southern, and western outskirts. His well digested report and military map of Nebraska and Dakota have been of great value, both in the development of the country and for the scientific information that they contain.

After nine years of this varied and active service, Lieutenant Warren was ordered in 1859 to West Point, in the department of

mathematics, and he remained there until the outbreak of the civil war.

He brought to the strife an intellect fitted for high command, a courage which knew no fear and shrunk from no responsibility, a judgment ripened by responsible duties, an earnest patriotism free from fanatical bias, and an energy so indomitable that it carried his delicate frame through labors and exposures which broke down many men of stronger physique. Like most soldiers of conscious ability, he despised the vulgar arts and clap-trap which form the stock in trade of coarser natures; and his magnanimity to the vanquished equalled his stubborn persistance during the contest.

The position of Lieutenant-Colonel of the Fifth New York Volunteers was very early tendered to Lieutenant Warren; and having received a leave of absence with permission to accept a volunteer command, he was mustered into the service in that grade on May 14, 1861. The regiment, as soon as organized at Fort Schuyler, was ordered to report to General Butler at Fortress Monroe, and at once proceeded south by sea. It came under fire for the first time in the affair at Big Bethel, fought on June 10, 1861, where Lieutenant-Colonel Warren was conspicuous for coolness and good judgment. He was the very last to leave the field, having remained to rescue at the risk of his life the body of his friend, Lieutenant John T. Greble, Second Artillery—the first in our little band of regular officers to die for the cause of National unity. Warren went back with about ten men, on learning of his death, and leaving them under cover advanced alone and carried the body in his arms to an abandoned limber, which was then drawn off by the party.

On August 31, 1861, he was promoted to be Colonel of the Fifth New York. During the remainder of the year the regiment was stationed in Baltimore, where it was engaged in constructing the large earthwork on Federal Hill, and in receiving the thorough drilling which made it confessedly one of the very best regiments in the service.*

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*The Prince de Joinville in 1862, in writing of our volunteer army, said:

"Sometimes an officer of the regular army, desirous of distinguishing himself, and having enough of influence in his State, raised a regiment and obtained from it an admirable result. Thus, a young Engineer Lieutenant named Warren was marvellously successful with the Fifth New York Regiment, of which he was the Colonel. That regiment served as Engineers and Artillery in the siege of Yorktown; and having again become Infantry conducted itself as the most veteran troops at the battles of the Chickahominy, where it lost half its force."

When the Army of the Potomac moved to the Peninsula in the spring of 1862, the Fifth New York accompanied it. Before Yorktown it formed part of the siege train under the command of General Barry, Chief of Artillery, Colonel Warren in addition doing much personal reconnoitering of the enemy's lines as an Engineer. The regiment was in camp near General McClellan's headquarters; and no officer who witnessed the daily dress parades of his 800 soldiers in brilliant zouave uniform and splendidly drilled, could fail to recognize the skill of the young Colonel as a disciplinarian and regimental commander.

After the advance began (on May 24), Colonel Warren was assigned to the command of the Third Brigade in Sykes' Division of the Fifth Army Corps, consisting of his own and two other Infantry Regiments, a Cavalry Regiment, and a Light Battery. With this Brigade he covered the extreme right of the army; and took part in the capture of Hanover Court House; the pursuit of Stuart's cavalry after the brilliant raid round our rear (marching his Infantry 43 miles in 37 hours); the battle of Gaines' Mill, where he was slightly wounded, and his horse was twice shot under him; the affair at Malvern Hill on June 30, and the great battle there of the following day. The Brigade lost 60 or 70 men killed and 150 wounded in these operations, chiefly in the battle of Gaines' Mill, and Colonel Warren was highly commended for gallantry and good conduct.

After leaving the Peninsula, Colonel Warren's brigade was landed at Aquia Creek and took part in the movements of the Fifth Corps to reinforce General Pope. In the desperate battle fought near Manassas, On August 30, 249 out of the 490 soldiers of his own regiment were killed and wounded, and his bull-dog tenacity did much to cover the withdrawal of the remnants of the Corps.

Recommended by his superior officers, and urgently pressed by General McClellan, he was appointed on September 26, 1862, Brigadier General of Volunteers for distinguished conduct at the battle of Gaines' Mill. He had in the meantime been engaged with his brigade in the Maryland campaign and the battle of Antietam. His command passed through Harper's Ferry on November 1, marched to Falmouth, and took part in the Rappahannock campaign and the battle of Fredericksburg.

While the army lay in the winter cantonments General Warren did much individual work in reconnoitering and correcting the maps; and finally, on February 2, 1863, he was ordered as Chief

Topographical Engineer to the staff of General Hooker, who had just assumed command of the Army of the Potomac. The two Corps of Engineers were consolidated by Act of Congress approved March 3, 1863; and on June 8, General Warren was appointed Chief Engineer of the Army of the Potomac, acting in that capacity until August 12. During the six months in which he thus served on the staff, his papers prove that he discharged highly responsible duties. In the Chancellorsville campaign he took a gallant part in the action of Orange Pike, the storming of Marye's Heights, and the battle of Salem.

Few better illustrations of the intensity of life at this time can be given than the circumstances attending General Warren's marriage with Miss Emily F. Chase of Baltimore, then residing with her father in that city. Hastening from the front, he arrived at 9 a. m. on June 17; was married at noon; and on the 20th was back at his post actively engaged in the movement toward Gettysburg. The life-long sympathy and love of his noble wife lightened many hours of despondency under the burden of wrongs which otherwise might have proved unendurable to a man of his proud and sensitive nature.

At Gettysburg, where he was slightly wounded, General Warren brilliantly distinguished himself as an engineer staff officer. On the second day of the battle (July 2d), after a personal examination of the right of the line near Culp's Hill, where an offensive movement on our part was in contemplation, he was drawn to the left by Longstreet's furious attack. At the moment when Hood, having outflanked Sickle's Corps, was thrusting forward his right, Warren had fortunately reached the bold and rocky spur called Little Round Top—the key to the whole Union position. It was entirely undefended, although occupied as a signal station. Appreciating the vital importance of the Confederate movement, Warren ordered the signal men, who were preparing to avoid capture by flight, to continue waving their flags and thus preserve a semblance of occupation while he hurried for troops. He soon encountered the head of Sykes' column hastening to support Sickles and assumed the responsibility of diverting Vincent's brigade to seize and occupy the hill, using General Meade's name as his staff officer. How gallantly this movement was executed in a desperate hand-to-hand conflict, in which Vincent and Weed, O'Rourke and Hazlitt, and hundreds of other soldiers in blue laid down their lives, is a matter of history. It was one of the many turning

points of this, the supreme battle of the war, and but for Warren's military coup d'æil and prompt acceptance of responsibility, Gettysburg might now be known as the grave of the Union.

The passage of the Potomac after the battle of Gettysburg afforded an illustration of the curious expedients upon which the success of engineer operations often depends. The pontons had been scuttled, and, as was supposed at the time, destroyed, in the preliminary operations of the campaign. It now became necessary to patch and repair the shattered boats at once; and at General Warren's personal suggestion, this was done successfully with cracker-boxes obtained from the Subsistence Department.

On August 8 General Warren was appointed Major-General of Volunteers, to date from May 3, when he had distinguished himself with General Sedgwick's column at the storming of Marye's Heights and the battle of Salem. On August 11 he was assigned to the temporary command of the Second Corps. He had thus in two years, without influence other than the recommendations of his commanding officers, fairly fought his way from the command of a regiment to that of an army corps.

His first important service in this grade occurred in Lee's flank march upon Centreville, in October, 1863. On the night of the 13th, when the Confederate army reached Warrenton, the Second Corps, forming the rear guard of the Army of the Potomac, bivouacked at Auburn, distant only about 5 miles. Neither army commander knew accurately the position or line of march of the other, but both were maneuvering to bring on a decisive battle. The march ordered by General Meade for the Third, Fifth, and Second Corps on October 14 lay along the Alexandria Railroad toward Centreville, Lee's supposed objective. During the night of October 13 General Stuart, with a brigade of cavalry, found himself entangled among the Second Corps, and just before daylight opened suddenly with artillery upon the camp fires of Caldwell's division. An infantry attack by General Ewell followed promptly from the opposite direction. Although repelled, these attacks delayed the Second Corps; so that when it reached Bristoe Station a small gap existed between its leading division (Webb's) and the rear of the Fifth Corps, next in advance. The head of General A. P. Hill's Corps struck this gap and immediately attacked. The moment was critical, but General Warren, who was on the spot, was equal to the emergency. With the utmost promptitude his two leading divisions were faced to the left and hurried forward under

fire to seize the railroad embankment and cut, thus securing a strong line. A sharp attack by Gen. Hill in line of battle was vigorously repulsed, and 450 prisoners, 2 stands of colors, and 5 pieces of artillery, were captured. Warren held this position for some hours with a force of less than 8,000 men, confronting the whole of Hill's Corps (numbering about 17,000 men), gradually increased by the whole of Ewell's Corps during the afternoon. At dark he was reinforced by part of the Fifth Corps; and during the night was ordered to continue his march toward Centreville. He crossed Bull Run about 4 a. m. with his wounded and captures, having in 24 hours twice repulsed the enemy in superior force and marched over 25 miles. The total loss of the Second Corps in killed and wounded was 433 officers and enlisted men; and of the Confederates, in killed and wounded, 782 officers and enlisted men. General Humphreys, then Chief of Staff of the Army of the Potomac, writes: "The handling of the Second Corps in this operation, and the promptitude, skill, and spirit with which the enemy was met were admirable, and might form an excellent model for the conduct of the rear guard."

General Meade, in an order published to the Army, said: "The skill and promptitude of Major-General Warren and the gallantry and bearing of the officers and soldiers of the Second Corps, are entitled to high commendation."

General Warren's next conspicuous service was in the Mine Run movement of November, 1863. On the 29th, with his own Corps and a division of the Sixth, he reached a position on the extreme right of the enemy, which, after careful examination, he reported favorable for assault. General Meade ordered a combined attack, to begin by an assault by Warren's command (reinforced during the night by two divisions of the Third Corps) at 8 o'clock on the following morning. At daylight General Warren discerned that the opportunity had passed; for during the night reinforcements had arrived and had so strongly entrenched the position as in his belief to render its capture hopeless. He had the moral courage to assume the responsibility of suspending the movement; and General Meade, after an immediate personal inspection, confirming his judgment, the useless effusion of blood was spared. This action of a young General in temporary command of a Corps, displaying a willingness to sacrifice his own future prospects rather than squander the lives of his soldiers, illustrates the character of the man.

At the reorganization of the Army of the Potomac into three Corps for the Richmond campaign, General Warren was assigned by the President (March 24, 1864) to the permanent command of the Fifth Corps. Space is lacking to trace his personal career during the year in which he held this high command. It will find a place in every true history of the war. Suffice it to say that he played a conspicuous and honorable part in the battles of the Wilderness, Spottsylvania, North Anna, Bethesda Church, Cold Harbor, and especially in the numerous battles around Petersburg. Everything that ability and skill, and personal gallantry and devotion to the cause could do, Warren did; and he received the highest reward of a successful General—the confidence, the love, and the support of his soldiers. This latter is no vague statement; but is based upon the personal knowledge of the writer at the time, confirmed by many letters from officers of distinction now on file. Indeed the wildly enthusiastic greeting of the whole Fifth Corps on its return through Petersburg, establishes its truth beyond cavil.

We come now to the battle of Five Forks. The operations which culminated in this decisive action are fully established by sworn testimony before the court of inquiry which General Warren, after nearly fifteen years of persistent effort, succeeded in obtaining from the President. Space permits a brief summary only of the more salient points; but history can not now fail to do him ample justice.

At sunset of March 31 the Fifth Corps occupied the extreme left of the Union position; and General Sheridan's cavalry was at Dinwiddie Court House—distant about 5 miles to the left and rear. Both had been severely attacked during the day, and the latter was still confronted by infantry and cavalry. At 8.40 p. m. General Warren himself suggested that he be allowed to move in force against the rear of the enemy operating against General Sheridan. On his own responsibility, as early as 5 p. m., he had dispatched a strong brigade with orders to attack that force; and in consequence of this movement the Confederates withdrew during the night from General Sheridan's front.

About 7 a. m. of April 1, the Fifth Corps and the cavalry effected a junction, and under command of General Sheridan prepared for a combined attack upon the enemy—then at Five Forks, a detached position about four miles to the westward of the Confederate main intrenched line before Petersburg. The country was much wooded. The cavalry was early disposed along the enemy's front, the

Fifth Corps (12,000 men) being left massed at J. Boisseau's until ordered forward about 1 p. m. About 4 p. m. it had advanced about 2½ miles, and formed near Gravelly Run Church ready to assault.

General Sheridan's purpose was to crush and turn the Confederate left flank with the Fifth Corps, at the same time assaulting their line of battle in front with his cavalry.

The Fifth Corps advanced as directed by General Sheridan, Ayres' division on the left, Crawford's on the right, and Griffin's in reserve. The indicated point of attack lay too far to the right. Ayres soon received a sharp fire on his left flank from the return which formed the extreme left of the Confederate position. He promptly changed front, assaulted and finally handsomely carried this angle, taking many prisoners. This movement left the other divisions advancing in air with only a cavalry force to oppose them, and Warren hastened in person to change Crawford's direction to the left, having previously sent orders to Griffin to move to his left and come in on the right of Ayres. The country was rough and wooded, and the position of the enemy had been supposed by General Sheridan to extend much more to the eastward than was actually the case. Hence the primary importance of these movements, in order to bring the whole Fifth Corps into action.

In this difficult task Warren was everywhere—first with Crawford's division, establishing the new line of advance; then with Griffin, directing him upon the enemy lying along the west side of the Sydnor field—whose exact position he had just discovered by drawing their fire upon himself; then to Ayres, finding him in possession of the angle with many prisoners; then back to Crawford, and conducting the advance through the woods so as continually to outflank the enemy in his attempt to form new lines to cover his natural retreat (the Ford Road) and to hold the position at the forks. Finally Crawford's division, still accompanied by Warren, and having swept everything before it, found itself on the east side of the Gilliam field, but somewhat disorganized by the fighting through difficult woods. Confronting it on the west side was a new and last line of the enemy slightly intrenched.

Here a pause occurred, and personal magnetism seemed called for to lead on the troops who for the moment had lost their organizations in the confusion. Warren having discharged the more pressing duty of directing the whole force of his Corps upon the enemy, now found time to yield to his natural impulse. He seized

his headquarters flag, rode into the opening, and calling on the color-bearers to advance, led the charge. His horse fell dead under him close to the enemy's lines; an orderly by his side was killed; and his own life was probably saved by the gallant act of Colonel Richardson, Seventh Wisconsin, who sprang between him and the enemy, receiving a severe wound. This charge put an end to all resistance. Surrounded by his captures and flushed with victory, Warren sent back a staff officer to report to General Sheridan and ask for further orders.

These orders came in writing. They relieved him from the command of his Corps and ordered him to report to General Grant.

If the bullet which killed his horse had pierced the heart of the rider, Warren, like Wolf dying upon the Heights of Abraham, would have gone down in history the hero of the battle. This order, more cruel than the bullet, doubtless caused his death after seventeen years of suffering which intimate friends who understood his sensitive organization can alone appreciate. It is pitiful that one of his last requests was to be laid in the grave without the usual military ceremonial, without soldierly emblems on his coffin, or uniform upon his body. The iron had entered his soul.

General Grant, on April 3, assigned him to the command of the defenses of Petersburg and the South Side Railroad, and on May 14 he was transferred to the important command of the Department of Mississippi; but on May 27, as soon as he felt assured that the fighting was over, he resigned his volunteer commission of Major-General, and returned to duty as Major in the Corps of Engineers. He received several brevets in the regular army for gallant and distinguished services in battle, but with such a record as his they need not be named.

Of his services in the civil branches of his profession since the war, I shall here say nothing. They covered a wide range of subjects, and would give him prominence among eminent engineers in any country. The Corps order of General Wright, announcing his death, contains the following fitting tribute to these labors: "In scientific investigations General Warren had few superiors; and his elaborate reports on some of the most important works which have been confided to the Corps of Engineers are among the most valuable contributions to its literature."

The lives of few graduates more perfectly illustrate the fruits of what we are proud to call West Point culture than that of General Warren. Everything with him was subordinated to duty,

and he had put forth his whole strength in whatever he had to do. His tastes were cultivated and refined, and his reading in both literature and science was extensive. A man of warm affections and sympathetic nature, he was ever ready to listen to the cry of distress. Even after his long experience in war the misery of the wounded and the severe hardships of all his soldiers, in some of the winter movements south of Petersburg, so touched his heart that he wrote to his brother: "I do not feel it much in my own person, but I sympathize so much with the suffering around me that it seems at times I can hardly endure it." He is now peacefully at rest beyond the reach of praise or censure; but his memory is a sacred legacy to West Point and to the Army of the Potomac. There is no nobler name upon either roll.

HEADQUARTERS CORPS OF ENGINEERS,
UNITED STATES ARMY,
Washington, D. C., August 9, 1882.

(GENERAL ORDERS, NO. 5.)

It has become the painful duty of the Brigadier General Commanding to announce to the Corps of Engineers the death of a brother officer, Lieutenant Colonel *Gouverneur K. Warren*, Brevet Major General, United States Army, who died at Newport, R. I., yesterday.

General *Warren* was graduated from the Military Academy and promoted to the rank of Brevet Second Lieutenant in the Corps of Topographical Engineers, July 1, 1850. He served as Assistant Engineer on the topographical and hydrographical survey of the Delta of the Mississippi, 1850-'52, and to the Board for the improvement of Canal around the Falls of the Ohio, 1852-'53; in charge of Surveys for the improvement of Rock Island and Des Moines Rapids, Mississippi River,1853-'54; in compiling the General Map and Reports (conjointly with Captain, now General *A. A. Humphreys*) of Pacific Railroad Explorations, 1854; as Chief Topographical Engineer on Sioux Expedition, 1855, being engaged in the action of Blue Water, September 3, 1855; in charge of reconnaissances in Dakota Territory, and making Map and Report of same, 1855-'56, and in Nebraska Territory, 1856-'57, and preparing Maps and Reports thereof, 1857-'59.

He was Assistant Professor of Mathematics at the Military Academy, 1859, and Principal Assistant Professor, 1859-'61.

He entered upon his distinguished service in the late civil war (1861-'66) in the Department of Virginia, as Lieutenant-Colonel

of the Fifth New York Volunteers, being engaged in the action at Big Bethel Church, June 10, 1861. He was engaged on the defenses of Baltimore, and constructing Fort on Federal Hill, 1861-'62, being temporarily detached on expedition to Northampton and Accomac counties, Va., 1861; in the Virginia Peninsula Campaign (Army of the Potomac) 1862, being engaged in the siege of Yorktown, April 11-May 4, 1862, and in command of Brigade, May 24, 1862; skirmish on Pamunkey River, May 26, 1862; capture of Hanover Court House, May 27, 1862; battle of Gaines' Mill, June 27, 1862, where he was wounded; repulse of Wise's Division at Malvern Hill (in command), June 29, 1862; battle of Malvern Hill, July 1, 1862, and skirmish at Harrison's Landing, July 2, 1862.

In the Northern Virginia Campaign, 1862, he was engaged in the battle of Manassas, August 30, 1862, and skirmish near Centreville, September 1, 1862. He was in command of Brigade (Army of the Potomac) in the Maryland Campaign, 1862, being engaged in skirmishes and battle of Antietam, September 15-17, 1862; skirmish with the enemy's rear guard on the Potomac, September 19, 1862; and march to Falmouth, Va., 1862. In the Rappahannock Campaign, 1862-'63, he was in command of Brigade till February 4, 1863. He then became Chief Topographical Engineer of the Army of the Potomac, and was engaged in the battle of Fredericksburg, December 13-16, 1863; making reconnaissances, 1862-'63; action on Orange Pike, May 1, 1863; storming of Marye Heights, May 3, 1863, and battle of Salem, May 3-4, 1863, and as Chief Engineer of the Army of the Potomac, June 8-August 12, 1863.

In the Pennsylvania Campaign he was engaged in charge of the re-embarkation of stores at Aquia Creek, 1863; reconnaissance and battle of Gettysburg, July 1-3, 1863, where he was wounded; and construction of bridges, and making reconnaissances while pursuing the enemy, July-August, 1863.

He was in command of Second Corps (Army of the Potomac), from August 12, 1863, to March 24, 1864.

In the operations in Central Virginia, he was engaged in movement to Culpeper and the Rapidan, September 13-16, 1863; combat at Auburn and Bristoe Station (in command), October 14, 1863; skirmish at Bull Run, October 15, 1863, and at Kelly's Ford, November 8, 1863; movement to Mine Run, with heavy skirmishing, November 26-30, 1863, and demonstration upon the enemy across Morton's Ford, February 6, 1864.

He was in command of Fifth Corps (Army of the Potomac), from March 24, 1864, to April 1, 1865.

In the Richmond Campaign he was engaged in the battle of the Wilderness, May 5-6, 1864; battles about Spottsylvania, May 8-20, 1864; battles of North Anna, May 23-25, 1864; skirmish on Tolopotomy Creek, May 29, 1864; battle of Bethesda Church, May 30, 1864; battles of Cold Harbor, June 1-4, 1864; skirmish on White Oak Swamp, June 13, 1864; assaults on Petersburg, June 17-18,

1864; siege of Petersburg, June 18, 1864-April 2, 1865; Petersburg Mine assault, July 30, 1864; actions for the occupation of the Weldon Railroad, August 18-25, 1864; combat of Peebles' Farm, September 30, 1864; action at Chapel House, October 1, 1864; skirmishes near Hatcher's Run, October 27-28, 1864; destruction of Weldon Railroad to Meherrin River, December 7-10, 1864; combat near Dabney's Mill (in command), February 6-7, 1865; actions and movement to White Oak Ridge, March 29-31, 1865; battle of Five Forks, April 1, 1865.

He was in command of the defenses of Petersburg and Southside Railroad, April 3-May 1, 1865; in command of the Department of Mississippi, May 14-30, 1865, and was at New York City preparing Maps and Reports of his campaigns, June 20, 1865, to July 31, 1866.

General *Warren* was promoted successively from the grade of Lieutenant to that of Lieutenant Colonel, Corps of Engineers, and Major General, U. S. Volunteers. He received the brevets of Lieutenant Colonel, U. S. Army, "for gallant and meritorious services at the battle of Gaines' Mill," Va., 1862; Colonel, U. S. Army, "for gallant and meritorious services at the battle of Gettysburg," Pa., 1863; Brigadier General, U. S. Army, "for gallant and meritorious services at the battle of Bristoe Station," 1865, and Major General, U. S. Army, "for gallant and meritorious services in the field during the Rebellion," 1865.

Since the close of the war he has been Superintending Engineer of surveys and improvements of the Upper Mississippi and its Tributaries, 1866-'70; of survey of the Battlefield of Gettysburg, Pa., 1868-'69, and survey of the Battlefield of Manassas, 1878; of Rock Island Bridge across the Mississippi, 1870; of the fortifications of New London and New Haven, Conn., 1870-'74; of the improvement of certain rivers and harbors on Long Island, 1870-'74; of construction of Block Island Breakwater, R. I., 1870-'82.

He was a Member of Commission to examine Union Pacific Railroad and Telegraphic Lines, 1868-'69, and Member of many important Boards of Officers of the Corps of Engineers organized for the consideration of the plans and the execution of the works of the Corps, among which were the Board on Improvement of the Des Moines Rapids, 1867; Board on Bridge across Niagara River, at Buffalo, N. Y., 1870-'71; on Bridging the Ohio River, 1870-'71, and 1878-'82; on plan for docks constructed for Breakwater at Chicago Harbor, Ill., 1871; on the completion of Cincinnati and Newport Bridge over the Ohio, 1871; on the harbors of St. Louis, Mo., and Alton, Ill., and banks of the Mississippi, 1872; on Bridging the channel between Lake Huron and Lake Erie, 1873; on Ship Canal from the Mississippi to the Gulf of Mexico, 1873-'74; to examine the St. Louis bridge across the Mississippi, 1873; on the reclamation of the Alluvial Basin of the Mississippi, 1874-'75; on Mississippi bridges between St. Paul, Minn., and St. Louis, Mo., 1876, and on the improvement of the Mississippi from the Falls of St. An-

thony to Rock Island Rapids, 1878. He was engaged in the survey of the Battlefield of Groveton, Va., and in the preparation of campaign maps of certain operations in 1862-'63 of the Army of the Potomac in Virginia.

He was appointed a Member of the Advisory Council of the Harbor Commissioners of the State of Rhode Island, 1878.

In 1870, General *Warren* was assigned to the charge of the surveys and improvements of various rivers and harbors in southeastern Massachusetts; and in Rhode Island and Connecticut, on which duty and in the supervision of the construction and repair of the fortifications of New Bedford, Mass., of Narragansett Bay and of Newport, R. I., he remained until the time of his death.

In scientific investigations General *Warren* had few superiors; and his elaborate reports on some of the most important works which have been confided to the Corps of Engineers are among the most valuable contributions to its literature.

In the field, in the late civil war, he was a brave and energetic officer, and in the high command to which he attained by his patriotic valor and skill, he merited the admiration of the army and the applause of his country.

He was kind and considerate in all the relations of life, and his family in its affliction will have the hearty sympathy of the Corps of Engineers.

As a testimonial of respect for the deceased, the officers of the Corps will wear the usual badge of mourning for thirty days.

By command of Brig. Gen. WRIGHT:

GEORGE H. ELLIOT,
Major of Engineers.

Equipment for District Photography

BY

Capt. W. G. Caples
Corps of Engineers

The simplicity of modern photographic apparatus, together with the camera's fidelity to detail, have led to the increasing use of photography on engineering projects. The intending purchaser of a photographic outfit or of supplies for such an outfit is confronted with a maze of equipment from which the most suitable articles can be selected only by a series of comparative tests which may cover the better part of a year. The writer has just completed a series of tests for military outfits and has been requested to suggest some outfits especially adapted to district work and to embody a description of them in this article.

At the outset, be it said that all standard lines of equipment are good, that each maker excels in some specialties, but that no maker is the leader in all lines, and that the prices of articles of the same quality are practically uniform regardless of make. Names and brands are freely mentioned, with the sole idea of selecting the best articles without regard to make and of combining them into suitable outfits. Nothing is claimed for the selections, beyond the fact that they are the ones indicated by somewhat limited tests and experience.

The widest need seems to be for a small outfit, simple enough for the beginner, good enough for clear results of medium size and angle, strong enough to withstand moderate abuse, very compact, equally adapted for either rolls, films, or plates, cheap enough to be placed on each active project, and costing for operation so little that the number of views taken may be liberal. The conditions imposed necessitate a folding hand camera, constructed almost entirely of metal, with a focal capacity preferably more rather than less twice the diagonal of the plate, a broad, rigid, sliding bed and lens board support, the usual adjustments, an automatic stop for universal focus, and using the popular $3\frac{1}{2}$ by $5\frac{1}{2}$ inch plate. The optical equipment should consist of a low-priced moderate aper-

ture, symmetrical doublet, anastigmat lens set in a compound shutter with F system graduation for its diaphragm, giving exposures up to 1-250th of a second, operated with a wire release and fitted with a compensating filter.

The camera proposed is the only one offered to meet all these conditions. Its tests were entirely satisfactory, but the makers state that they expect to place even a better instrument on the market inside of the next year.

The advanced amateur uses a small camera and secures prints by enlargement. The time required for enlarging and retouching—since defects enlarge also—make this process of doubtful value as the sole reliance for large views in a district. The time of an employee will soon amount to a sum that represents the cost of a very large outfit if the enlarging method is followed. For contact printing, a large image and medium angle can be provided only by a long focus lens and a large plate. This follows from the fact that the size of image at a given distance is in almost direct ratio to the focal length employed, while the useful angle is practically twice the angle whose tangent equals half the base of the plate divided by the focal length.

The solution seems to be to place in each district one first-class central outfit giving large size images over a useful angle of 40° to 50°, and as many small local outfits, not less than one, that may be required for progress views, preliminary examinations and similar work.

Three sizes of plates, 5 by 7, 6½ by 8½, and 8 by 10 have been considered for the central outfit. The 5 by 7 size may be had either in the hand or view types of camera, while the larger sizes are adapted only for the view camera. A lens should have an equivalent focal length about equal to the diagonal of the plate, but, with a hand camera, a 7-inch lens is about as large as can be used on a 5 by 7 outfit. The different outfits compare about as follows:

	Angle.	Relative size of Image.	Relative cost complete.	Approximate weight of view section only.
				Lbs.
1. Local outfit, 6.5″ lens	46°	1.00	1.00	6
2. 5×7, Hand Camera, 7″ lens	53°	1.08	2.22	18*
3. 5×7, View Camera, 8.25″ lens	46°	1.27	2.15	20
4. 6.5×8.5, View Camera, 10.75″ lens	43°	1.65	2.33	25
5. 8×10, View Camera, 12″ lens	45°	1.85	2.64	30

*Inclusive of 12 plates.

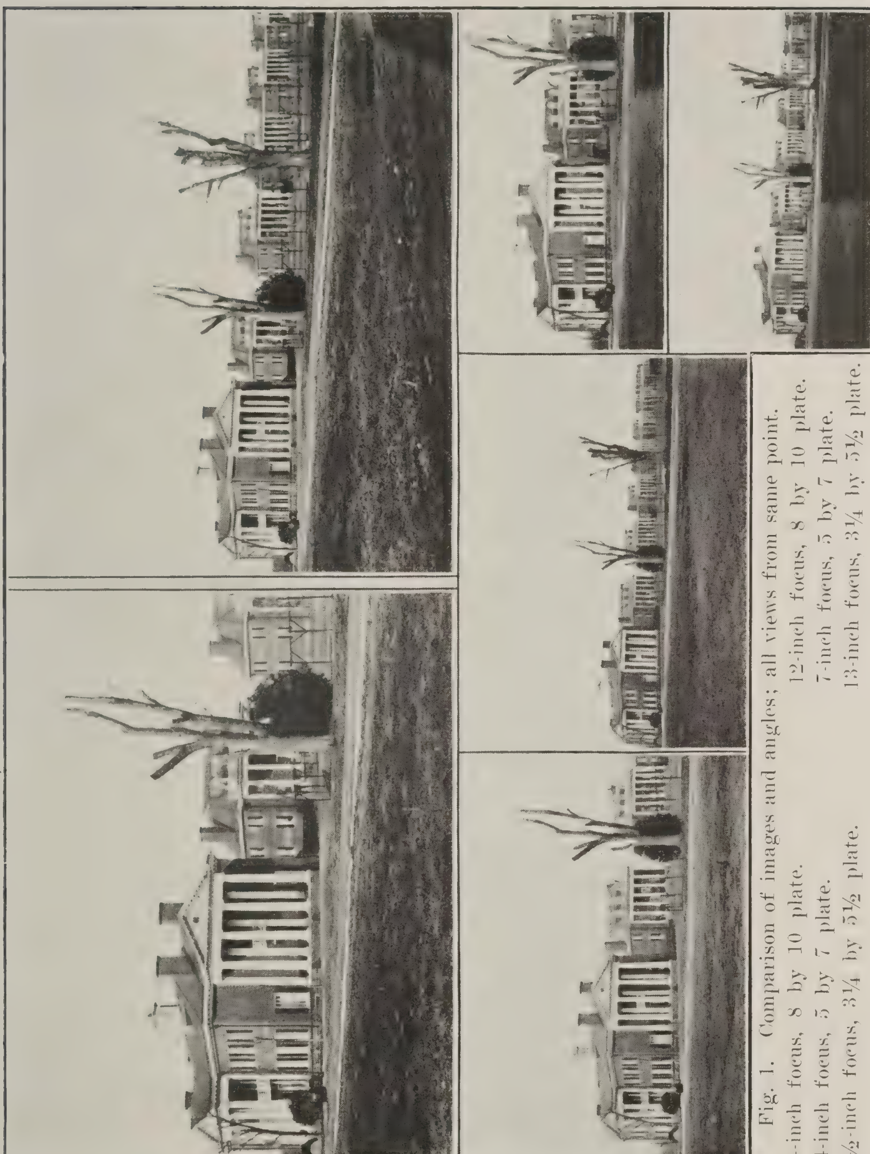

Fig. 1. Comparison of images and angles; all views from same point.

24-inch focus, 8 by 10 plate. 12-inch focus, 8 by 10 plate.

14-inch focus, 5 by 7 plate. 7-inch focus, 5 by 7 plate.

6½-inch focus, 3¼ by 5½ plate. 13-inch focus, 3¼ by 5½ plate.

The advantage of outfit No. 2 over No. 1 is so slight and its cost is relatively so high that only extreme compactness would commend its use. Outfit No. 3 has a decided advantage over Nos. 1 and 2, but is of the view-camera type, in which the 5 by 7 plate is distinctly inferior to the larger sizes. The 5 by 7 size is not considered suitable for a central outfit.

The addition of a lens board and flange enables either the No. 4 or the No. 5 outfit to do wide angle work, using the lens from local outfit, which will give about 65° on either the 6½ by 8½ or on the 8 by 10 plate. Neither the No. 4 nor the No. 5 is excessively heavy or bulky, and the cost of both is moderate. The relative costs are about as the relative sizes of image. The larger the image, the more useful the camera and the less the need of enlargement. The 8 by 10 can use a smaller plate, but the 6.5 by 8.5 can not use a larger one. All considered, the 8 by 10 seems the best size, although the 6.5 by 8.5 gives very good results.

Simplicity, rigidity, durability and adaptability (the essentials of a good camera) are the salient features of the camera recommended. One of this type has outworn more expensive cameras, even of the same make, and, at the Engineer School, is used in preference to newer and more expensive instruments.

A symmetrical doublet is really two lenses in one and has just twice the value of an unsymmetrical lens for engineering photography. To utilize such a lens, the camera must have a focal capacity about double the focal length of the doublet.

In the view field, as in most other fields, the anastigmat lens is supreme, but it does not seem necessary, for the work required, to pay the 25 per cent extra required for the extremely wide apertures. (F 6.3 and larger.) The engineer can pick his time and his light, leaving motion alone to be considered. The lenses proposed work at F 6.8. This aperture and a Hammer Red Label plate in clear sunlight, in the average latitude of the United States, at about noon of the winter solstice, will complete an exposure in about 1-250th of a second. At this speed, objects 100 feet or more from a 12-inch lens or 55 feet or more from a 6.5-inch lens, and moving not over 14 miles per hour will appear as if at rest. Furthermore, a higher speed calls for a focal plane shutter, since it is useless to put a fast lens on a slow shutter. The greater strength of the compound-shutter seems more desirable than the greater efficiency of the focal plane type. Automatic exposures with both are only approximate, while if the focal plane shutter

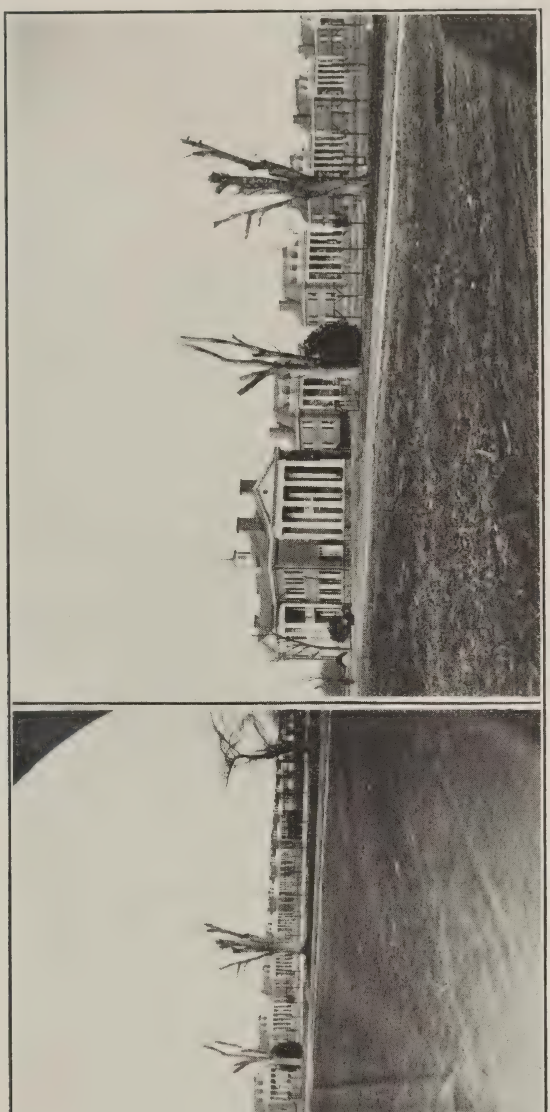

Fig. 2. Comparison of 6½-inch Syntor and 12-inch Dagor on 8 by 10 plate. Both views from same point. In the wide angle (left hand) view, the circle of clear definition is 8 inches in diameter.

be left long on the high tension or allowed to rust, its work becomes very uncertain. High speeds, or rather, large apertures are inseparable from small depth of focus. In most cases, the photographer will use tripod and filter and work at F 11 to F 23 to get depth and detail, reserving the speed work for such special occasions as require its use.

The lenses proposed have been selected after a comparative test of several leading makes. These tests indicate that while all confirm their maker's ratings, the lenses proposed have the maximum covering power and uniformly exceed the cautious rating given by their makers. The lens on the local outfit seems to be the best of the cheaper anastigmats, while the lenses on central outfits, except for speed, certainly equal any other view lens, regardless of type or cost, and are about the lowest priced of the high-grade lenses.

Plates are necessarily better than films. A lens focuses objects in different places only because it brings the images of all points so nearly into the same plane that the circles representing the points are so small that the eye sees them as points. If the emulsion is not a plane surface perpendicular to the axis of the lens, some of these circles will be large enough to detect and the blurred image common to the back grounds of all large aperture work will be the result. The rigid glass of a plate, once made plane, remains so; but the flexible celluloid of the film must be pulled or pressed to a plane surface. The larger either plate or aperture, the more apparent is the difference. A rough comparison of negatives leads the writer to believe that a good plate has more gradation (more grades of light shadow) than the roll films. Upon gradation depends detail in shadows and monochrome subjects. Detail is specially desirable in engineering work.

Plates are from 20 per cent to 30 per cent cheaper than roll films, and soon make the difference felt in the bill. Consider for one moment that a single good flash of actinic light will destroy all sensitive material within its reach, and it is evident that claims for "day-light-all-the-way" must be taken with a pinch of salt. The prudent man will either use a dark room or work at night in a darkened room, which amounts to the same thing. In either case, plates can be used as well as films. A small supplementary outfit is strongly recommended as an addition to local outfits to fit them for plates. The only change involved is one of method of development, the factorial method described below being con-

sidered best for the small number of plates used. If daily progress views are wanted, either the same method must be followed with films or an entire roll used each day.

Many illustrations in the MEMOIRS come from an old outfit identical with the view camera outfits proposed. The lens has been sadly mistreated until the cement has run, and the camera has been racked by hard field service. The quality of results speaks for the highest standard of the equipment. The views,

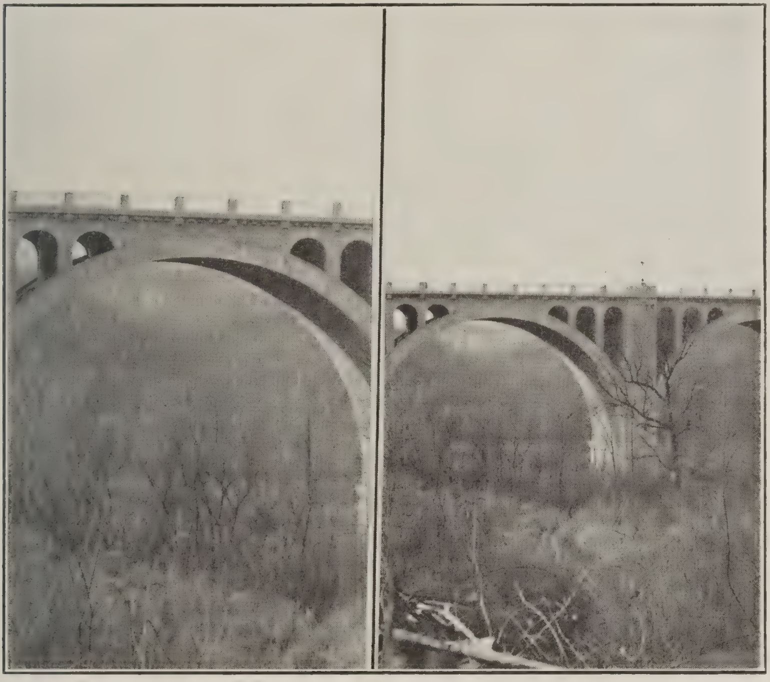

Fig. 3. Back combination alone and complete, 6½-inch doublet on 3¼ by 5½ plate. Both views from same point.

from a local outfit, accompanying this article were all made as tests under the most unfavorable conditions that could be had. In all of them, the lens has been made to do work equivalent to covering an 8-inch circle.

Two developing methods, the factorial and the time-temperature, combine simplicity with certainty. Factorial development depends upon the fact that for a given developer the time of first appearance of any trace of image bears to the time of full development a fixed ratio, known as the Watkins factor. This method

is specially adapted to tray development but may, with less precision, be applied to open tank development. The time-temperature method depends upon the fact that for a given developer and a given plate, the time of development depends upon the temperature of the bath. Weak baths, exhausted by a single batch of negatives, are generally used. Since circulation of the bath is poor, too strict timing may cause under-development in weak baths. There is little danger of over-development. At the Engineer School, plates are left in a weak bath twice the computed time and emerge as clear brilliant negatives. Unless ice is available to control the temperature, a preliminary hardening in a 6 per cent solution of formaline is advisable.

All the outfits have been designed for the normal use of the time-temperature-tank method. For the application of this method, two patented specialties, the Kodak film tank for films and the Core frame for plates are easily leaders. The former is so generally known as to require no special description. The latter is scarcely known outside the professional trade. A Core frame is a brass, rubber-cement-protected rim, into which a plate is slipped as it comes from the holder and left until after the final washing and drying. One edge of the rim is extended to form a support on the edges of the tank. One's hands do not touch the solution, all danger of scratching the wet emulsion is eliminated, and the plate may be examined at any time.

Professional photographers escape the high cost of equipment and get articles exactly suited to their use by building their own. Wooden boxes, with tight joints and painted inside and out with "Mogul," "Probus," or even common asphaltum paint or lined with oil cloth, answer every purpose for which the more expensive tanks, trays, and washing boxes are intended.

DEVELOPING OR FIXING TANK, WASHING BOX, AND RUBY LAMP.

The developing and fixing tanks are exactly alike, and may be plain or may have grooves as shown to permit the photographer to place a cover on them and then leave the dark room. The washing box may be plain, water entering from a hose at one end and overflowing at the other. A better arrangement is for the water to enter through a perforated pipe under a perforated false bottom and overflow through weep holes near the top. A light-tight box, except for a glass double door, and provided with an incandescent bulb makes a ruby lamp. The inner door should

be of ground glass and the outer of ruby glass or a special glass for othochromatic plates, known as "isochrom safe light." The amount of light is regulated by two flaps of post-office paper arranged as shown. Trays are merely open boxes.

Machine printing gives certain results and enables all printing to be done before developing is commenced, avoiding any fatal trace of moisture being carried to the unprinted paper. A machine will pay for itself in the paper saved alone. Since the source of light is constant and at a fixed distance, there is no question about the time required for printing, no matter how long since the negative was used, if, when first made, a notation of the time be placed in one corner of the plate, *e. g.*, C. 3.5, meaning that the negative in the machine will give a Cyco print in 3.5 seconds. The printing quality of a fresh negative is generally

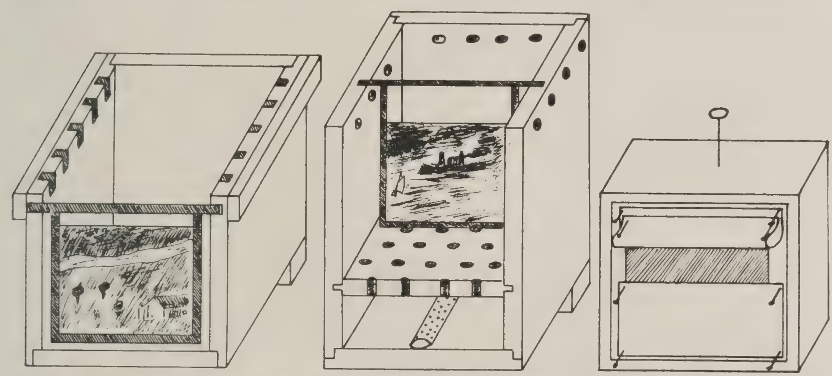

Fig. 4. Developing or fixing box, washing box, and ruby lamp. Showing also use of core frames.

determined by comparison with one of known printing quality, but a test slip may be used if desired. A glance at the printer in a central outfit will show how to build a home-made one for a local outfit that happens to have electric current available.

A print washer is needed only in cramped quarters, an open sink with vertical screened overflow answering the same purpose when space is available. A drying rack, consisting of sheets of white cheese-cloth on open frames, should be built. Their construction is so simple that ready-made ones are not on the market. Prints dried face down on cheese-cloth dry rapidly and without curl.

The following lists show each article in the different outfits, a brief specification, and the name of the maker.

VIEW SECTIONS FOR CENTRAL OUTFITS.

Plate Outfit, Sizes 6½ by 8½ and 8 by 10.

Camera equipment. "Empire State No. 2" camera, with "Combination" tripod (No. 2½ for 6½ by 8½, No. 3 for 8 by 10), 6 "Universal" double plate holders, "Professional" focusing cloth, in canvas case suitable for carrying entire view section. (Eastman. R. O. C. Division.)

Optical equipment. Dagor lens (No. 5 for 6½ by 8½, No. 6 for 8 by 10) in compound shutter F system graduated and with wire release, and with No. 2 Goerz compensating filter (Goerz).

Sundries. Wynne Infallible Exposure Meter with F system dial (Murphy).

Pocket magnifyer (Eastman Magnifying Glass for Focusing, No. 202 or equal).

Manual,† "The Book of Photography," Hasluck.

Note book, "Wellcome's Exposure Record and Diary," or "Wolfe's Record of Exposures."

This outfit may be had also in the 5 by 7 size, using the Dagor No. 3 lens, but it is believed that where considerations of bulk lead to the selection of so small a size of plate, the next outfit will be found preferable. The estimated costs of the different sizes are: 8 by 10, $135.00; 6½ by 8½, $115.00; 5 by 7, $100.00.

Plate Outfit, Size 5 by 7.

Camera, complete. "Manufoc-Tenax" size 5 by 7, No. 2 Dagor lens, No. 2 compensating filter, compound shutter F graduated and with wire release, 6 double metal "Tenax" plate holders, focusing screen, and 36 by 36 inch rubber focusing cloth (Goerz).

Tripod. Combination (Eastman).

Sundries. Wynne Infallible Exposure Meter, F system dial (Murphy).

Pocket magnifyer (Eastman Magnifying Glass for focusing, No. 202 or equal).

Note book, "Wellcome's Exposure Record and Diary," or "Wolfe's Record of Exposures."

Case. Canvas case with handle and clasp, built like suit-case and made expressly to hold above listed articles as compactly as possible.

Reference book. "The Book of Photography," Hasluck.

The estimated cost of this outfit is $115.00, but it is so compact that it may be slipped into a suitcase along with other baggage.

Plate Developing Section for Central Outfits.

(Size of articles marked * to be suited to size of plate used.)

*12 Core frames (Curry).

†Not carried in carrying case, but kept at office for reference.

(M) *1 developing tank, hard rubber (Curry).

(M) *1 fixing tank, hard rubber (Curry).

2 graduates, "Tumbler," sizes 8-ounce and 16-ounce (Eastman).

1 hydrometer (reliable). (Eastman.)

1 interval timer, Thayer No. 1 (Burke & James).

(M) 1 negative rack, Century (Eastman).

1 plate brush, rubber bound and set, camels hair, 3-inch (Eastman).

(M) 1 Ruby lamp (electric) "Studio" (Eastman).

1 scales and weights, "R. O. C. Balance" (Eastman).

2 stirring rods, hard rubber, 10-inch (Eastman).

1 thermometer (reliable). (Eastman.)

(M) *1 washing box, R. O. C. negative washer, suited to take size Core frame used (Eastman).

Articles marked (M) can be built by a carpenter at a distinct saving in cost and without any loss of quality or suitability. The cost, inclusive of articles marked (M) is estimated, viz: 5 by 7, $23.00; 6½ by 8½, $27.00; 8 by 10, $28.00. The cost of the items marked (M) is viz: 5 by 7, $11.10; 6½ by 8½ and 8 by 10, $14.35.

Printing Section for Central Outfits.

(Size of articles marked * to be suited to size of plate used.)

1 printing machine (electric), Ingento Rapid Printer, Style A (Burke and James).

*1 printing frame "Century" (Eastman).

(M) *1 print washer, Ideal No. 4 (No. 3 for 5 by 7). (Burke & James.)

1 squegee, Ingento, 12-inch (Burke & James).

1 trimming board, "New Monarch," 12-inch (Eastman).

*2 enameled trays, seamless (Eastman) or "Ingento" (Burke & James).

Article marked (M) can be replaced by washing sink built in dark room. The cost of this item is about $6.75 for 6½ by 8½ and 8 by 10, and $4.50 for 5 by 7. The estimated cost, inclusive of article marked (M) is $21.00 for 5 by 7, and $24.00 for 6½ by 8½ and 8 by 10 sizes.

Local Outfit Complete. 3.25 by 5.5 Roll Film.
View Section.

1 camera, complete. Special Film Tenax, size 3.25 by 5.5, with 6.5 inch Syntor lens, compound shutter graduated in F system, wire release and No. 2 compensating filter (Goerz).

1 tripod: folding head (Eastman).

1 exposure meter: Wynne Infallible, F system dial (Murphy).

1 note book: "Wellcome's Exposure Record and Diary" or "Wolfe's Record of Exposures."

The estimated cost of the view section is $59.00.

Developing and Printing Section.

1 developing tank: Kodak, $3\frac{1}{2}$-inch film tank, complete (Eastman).
1 graduate: Tumbler, 8-ounce (Eastman).
2 film clips, pair: Ingento, 4-inch (Burke & James).
1 manual: "The Modern Way in Picture Making." (Eastman).
2 printing frames: Century, 5 by 7 (Eastman).
2 push pins, dozen: Moore's, glass head.
1 ruby lamp: Ingento No. 9 (Burke & James).
1 scale, weighing: Ingento No. 2 (Burke & James).
1 shears, pair: Good quality 8-inch shears.
4 trays, seamless enamel: One each, $3\frac{1}{4}$ by $5\frac{1}{2}$, 5 by 7, $6\frac{1}{2}$ by $8\frac{1}{2}$, and 8 by 10, to nest (Eastman) or Ingento (Burke & James) or Elite (Murphy).

The estimated cost of the developing and printing section is $10.00.

Supplementary Plate Outfit.

1 case: Canvas, with clasp, to hold articles below listed (Goerz).
1 focusing screen: Tenax plate back (Goerz).
3 plate holders: Tenax, single, metal (Goerz).

The estimated cost of the supplementary outfit is $3.50.

Case for Outfit.

No case is regularly provided. The view section can be carried, along with other baggage, in a suitcase. If it is desired to pack the whole outfit into a single container, a suitcase will be found suitable.

The costs of all outfits are based upon the existing list prices and government discounts, both of which are subject to change.

It may be desired to enlarge a negative from a local outfit or a detail from a larger negative. The arrangements can be made by an ordinary carpenter and at a cost of about five or six dollars. Full directions applicable to enlarging with a local outfit are given in the advertising literature for enlarging with folding pocket kodak. Any arrangement that turns the camera into a magic lantern, excludes light other than that passing through the lens, and insures parallelism of plate and easel will serve.

ENLARGING ARRANGEMENTS FOR VIEW CAMERA.

The arrangement consists essentially of a frame to set under a window sash, a common mirror, an opening with a boxing a little larger than the camera back, a piece of heavy black cloth tacked to the boxing and long enough to stuff around the camera to shut out light leaks, a shelf with guides for the camera, parallel guides

on the floor, and an easel 20 by 25 inches or larger, painted *dull* white or evenly covered with drawing paper. The ground glass of the camera serves as a diffuser. A plate holder, with a suitable opening cut in the partition, serves as a negative holder. A reading glass and a ruby-glass lens cap complete the equipment.

The work of such an outfit, while not critical, produces no serious distortion in a view and, depending upon the negative, will give work as large as required. To insure that a negative

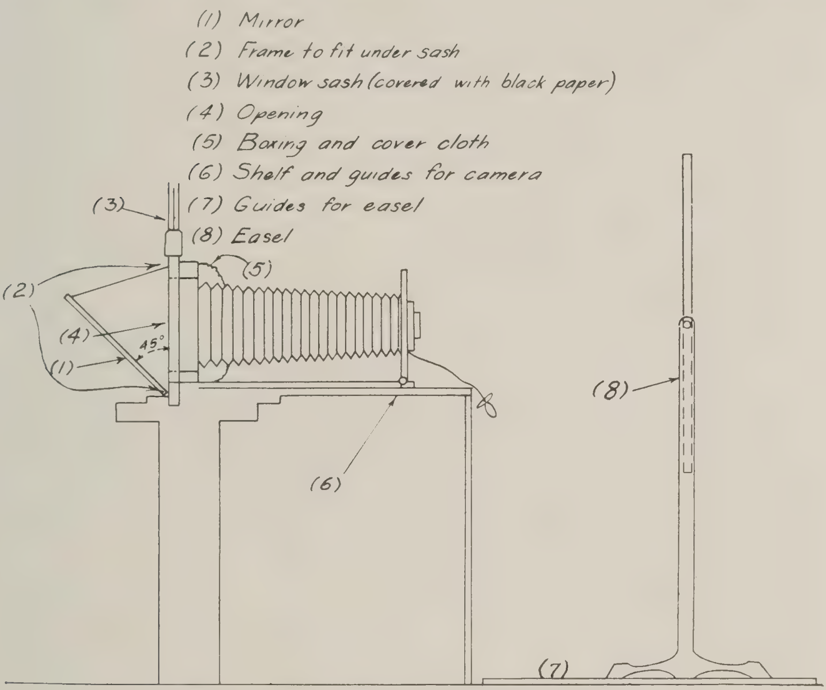

Fig. 5. Arrangements for daylight enlarging.

will enlarge well, a tripod, small stop, and medium or slow plate should be used.

Nowhere will care and ingenuity in design be repaid better than in the dark room. A badly arranged dark room is a prolific source of trouble and expense. Pyro or hypo, spilled on the floor and allowed to dry, will blow about and ruin materials to many times the cost of a properly arranged room. Once these destroyers are turned loose in a room they are very difficult to drive out.

ARRANGEMENTS FOR HANDLING PLATES.

All liquids should be handled in sinks or tubs having free open drainage. Water-proofed, reinforced cement is used for making these articles, unless more expensive ones are purchased. In a temporary room, such as would be found on a construction project, wood painted with asphaltum makes an inexpensive substitute. The general principles to be followed are that the arrangements should insure against spilling liquids on the floor and permit the different operations to be carried out in orderly sequence.

ARRANGEMENTS FOR HANDLING PAPER.

Of course, space is generally limited, so that the commodious arrangements shown must be modified to suit existing conditions.

Frames set in the windows and arranged with open ventilators, so that light enters vertically upward and makes four right-angle turns in dull black painted passage, and similar arrangements on the door panels will give ventilation without light. Moisture will quickly ruin material costing more than the ventilators.

For reliability, uniformity, and gradation, the Hammer plates can be recommended. High speed calls for the Red Label, but the finer grain of the slower plates makes them better for enlarging. The othochromatic plate should be used with the filter supplied in each outfit. Eastman N. C. film, however developed, and Eastman Speed film, except when tank developed, uniformly give excellent results. Tests of other roll film did not prove satisfactory.

It may surprise some to know that many prints in the finest galleries are on the same type of paper offered the amateur trade, while the demand among inferior professionals for "seconds" and even "thirds" of the amateur brands exceeds the supply. Each maker of developing paper produces twenty or thirty combinations of surface and contrast in different grades of the same brand to meet varying requirements, since papers of this class have little latitude in working. A single surface, matte or semi-matte, principally in the soft contrast, but with a little of the medium and hard grades will answer nearly every requirement. Cyco seems to be the professional favorite, but the writer's tests indicate that either Argo matte or Cyco plat are about equally suitable for views; the difference, if any, favoring the Argo. Either paper will do for enlarging also, but is painfully slow. Bromide is preferred for this class of work.

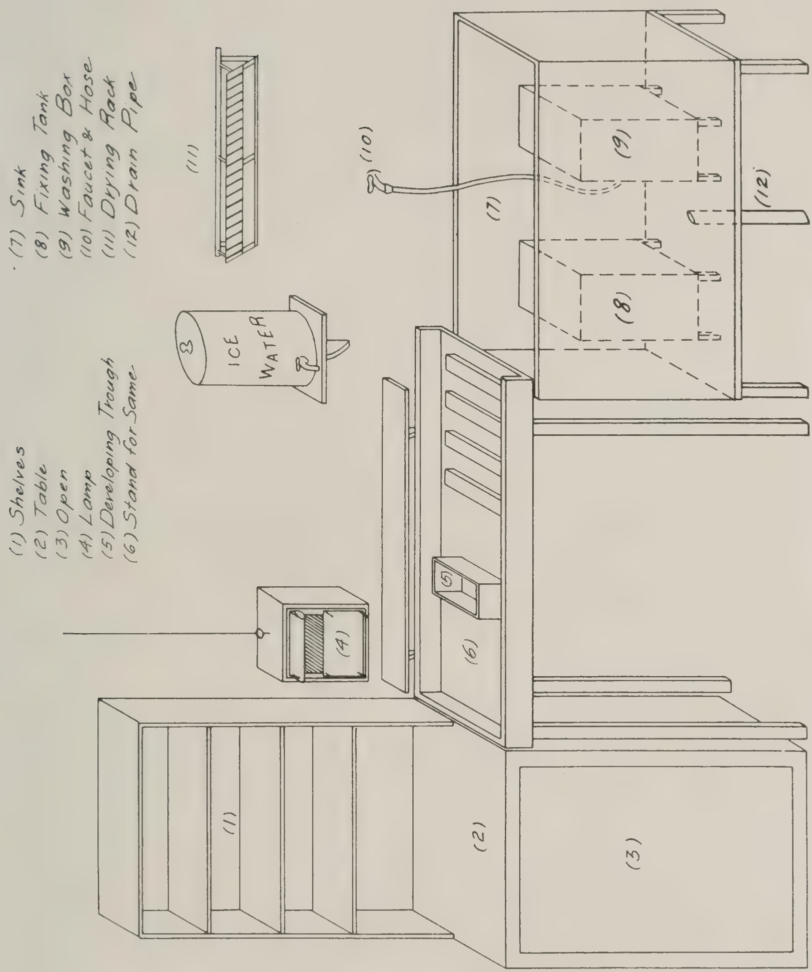

Fig. 6. Arrangement for handling plates.

Plates, films, and papers are guaranteed usually for one year from date of manufacture. The guarantee should not be more than one-fourth expired at date of delivery, as the fresher the better is the rule with such products.

After running the gamut of all classes of developers, professionals generally settle down to pyro in "A. B. C." formula for plates and edinol or metol in combination with hydroquinone for paper. So far as the writer has observed, the single developer formula, hereafter given, gives everything there is to be gotten out of either plates or paper. However, the manufacturers are constantly experimenting and publish their results in the directions with each package of sensitive goods and the manuals which they will send free upon request. An up-to-date list of these manuals should be on hand and their directions and formulæ carefully followed. Many manufacturers base their formulæ upon certain chemicals which they handle or recommend to protect the users of their goods. A good rule is to buy these chemicals.

For the man who uses but a single developer, either Rodinal or the following formula can be recommended:

Stock. (Dissolve in order given.)

Water	10 ounces	Hydroquinone	30 grains
Acetonesulphite (Bayer)	150 grains	Potassium Bromide (Crystal)	15 grains
Sodium Sulphite (dessicated)	450 grains	Potassium Carbonate (dessicated)	2 ounces
Edinol	60 grains		

Tray developer. Watkins factor 10.		Tank developer. 20 minutes at 65° F.		Paper developer. Bromide or Gas light.	
Stock	1 part	Stock	1 part	Stock	1 part
Water	4 parts	Water*	16 parts	Water	8 parts

A fairly complete list of expendable supplies commonly used is: absorbent cotton, brushes, chemicals, cheese cloth, glassware and glass, mounts, negative albums or preservers, note books, paste, plates or films, photo-preparations (a few), paper push pins, albums, refills for exposure meter, repairs, twine, varnish or paint.

Non-expendable supplies can ordinarily be limited to aprons, bath towels, chamois skin, and photo clips.

Negative albums for films and negative preservers (brown paper envelopes) and empty plate boxes for plates, with album of prints for an index, make a satisfactory filing system. A print should be filed back to back with each negative. Then a hurry call for a print does not take a man from other work to do the printing. A print pasted on the negative preserver is a help, but not essential where a record album index is used.

*Approximate. Should be tested for plate used.

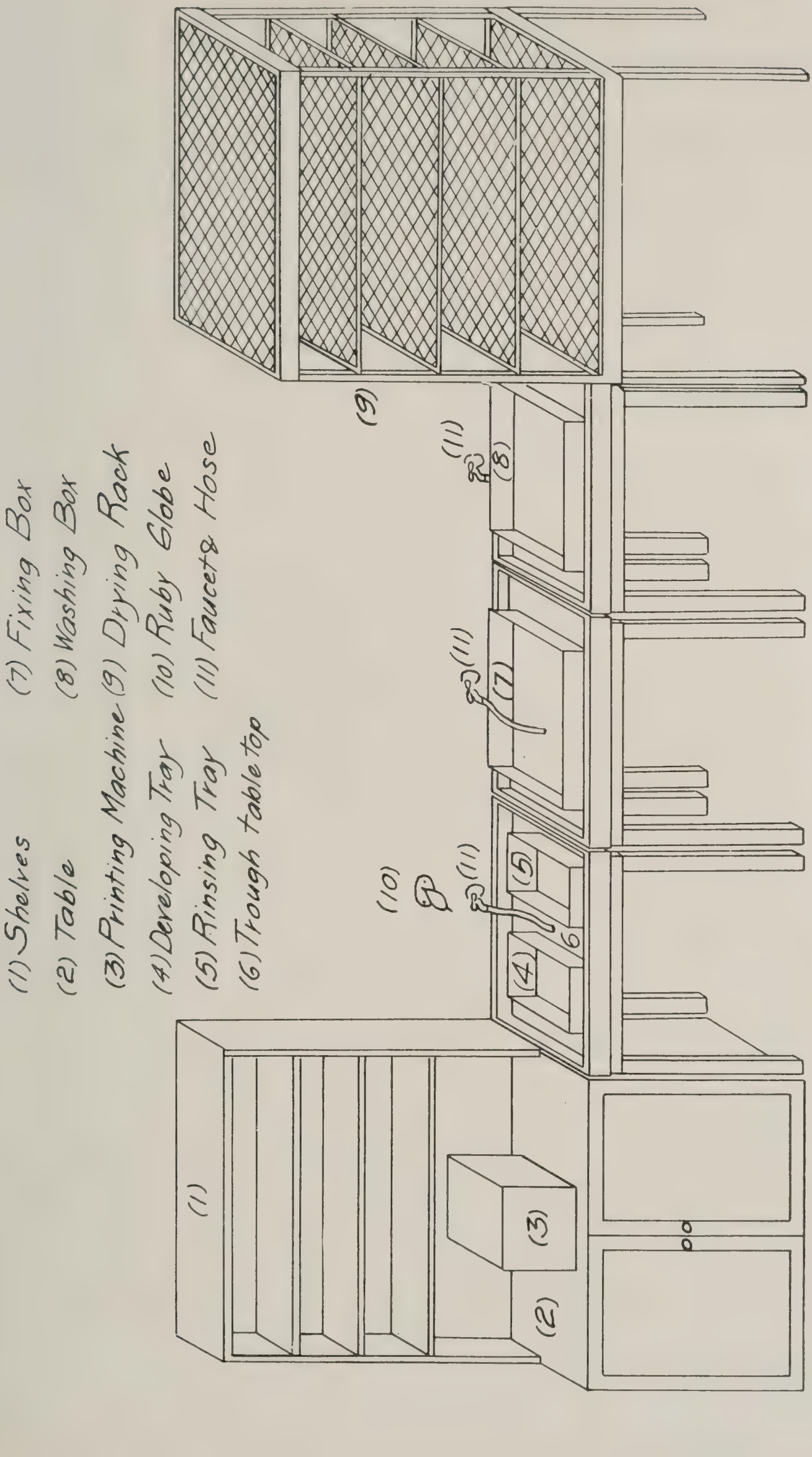

Fig. 7. Arrangements for handling paper.

Organization of the Services of Public Works in France*

Translated by

Maj. F. A. MAHAN
Corps of Engineers, Retired

(*Continued from No. 19*)

ORGANIZATION OF THE PERSONNEL.

The Constituant Assembly maintained the organization of the Corps of the Ponts et Chaussées as it has been set under the old régime by the decrees of December 31, 1790, and of May 25, 1791. But the direction of the Service was transferred from the Department of Finances to the Department of the Interior, which had among its duties: "The maintenance and execution of the laws concerning mines, mineral deposits and quarries, roads, bridges, and other public works, the preservation of navigation and rafting on rivers and towing along their banks, the direction of objects relating to buildings." (Art. 7 of the decree of May 25, 1791.)

Two separate divisions were formed at the Department of the Interior: that of Mines and that of the Ponts et Chaussées. The decrees of Fructidor 7, Year XII (August 25, 1804) and of August 7 and November 7, 1810, converted these divisions into directions and a royal ordinance of July 17, 1815, united them under a general direction, which was a first step toward a distinct department. The extension of public works brought about the creation of this department after 1830, and the ordinance of May 17, 1831, instituted the Department of Agriculture, Commerce and Public Works. It was separated in 1839, and the Department of Public Works was born. But it was suppressed by the second Empire, in 1853, and the Public Works were again made part of the Department of Agriculture and Commerce. It was only reestablished by the decree of July 17, 1869, since when it has always led an independent existence. It has charge of everything relating to

*Translation of "Organisation des Services des Travaux Publics en France," by M. Campredon. Brought up to 1912 by the translator.

the construction, repair, or maintenance of all the great works of public utility carried out by the State and it supervises all those which are granted to concessionary companies.

The entire Service of the Ponts et Chaussées belongs now to the second direction of this Department, the Direction of Roads and Navigation, which is itself subdivided in two divisions:

First division, Highways and Bridges; second division, Navigation.

The first division includes two bureaux.* The first directed by a chief of bureau with an assistant chief, two *rédacteurs*† and two *expéditionnaires*. It has charge of the national roads, their classification, construction, rectification, and maintenance; removal of buildings overlapping the sides of the roads; planting and care of trees.

Thermal roads in the Pyrenees; forest roads in Corsica, their completion and maintenance; the pavements of Paris, questions relating to the share of the State in the cost of maintenance; bridges, construction and maintenance of bridges which are parts of national roads; bridges and foot-bridges in Paris; legal matters relating to these various services.

2d Bureau. Office force: One chief of bureau, one assistant chief, two *rédacteurs*, and one *expéditionnaire*. It has charge of departmental roads; roadway police; hands employed on roads; national touring office; automobiles; declarations of public utility for works on departmental roads; regulation of sides of the roads; kilometric and intermediate marks; roadway police, regulation of traffic, etc.; dues for occupation of and standing on the roads; permits for the use of the roads otherwise than for travel; laying water and gas mains; hands employed on national roads; general studies relating to the maintenance of roads, rolling and tarring; road statistics; examination of the projects for communal works communicated by the Minister of the Interior; studies for the distribution of water in cities; purifying of sewage waters; grounds for

*The office forces here given are those of 1912; they may vary greatly according to work, circumstances, and available funds. They are composed mainly of clerks recruited by direct examinations in which legal rather than technical instruction is favored even for public works.

†*Rédacteurs* are clerks of a high order, able to prepare reports and various administrative papers from notes furnished, and to see that all documents are in proper legal form. *Expéditionnaires* are also clerks, but rather more of copyists.

target practice; military railways; national touring office; congresses of the road; general regulation of the circulation of automobiles; immatriculation; examination of drivers; claims; subventions for public automobile services.

The second division, that of navigation, includes four bureaus.

1st Bureau. Office force: One chief of bureau, one assistant chief, four *rédacteurs*, and two *expéditionnaires.* It has charge of seaports and of the light-house service; construction, improvement, and maintenance of commercial seaports; nautical commissions; operating and equipment; concession, construction, and operating of railways on the quays of seaports; dikes and works of defense against the sea, organization and working of the syndical associations for constructing these works; concessions of *polders** and of *lais* and *relais*† along the sea; marking and preservation of the maritime public domain; light-house and beaconage service of the coasts of France and Algeria, construction and maintenance of light houses, lighted and day beacons, sea and land marks; legal questions arising in regard to these services.

2d Bureau. Office force: One chief of bureau, one assistant chief, two *rédacteurs* and two *expéditionnaires;* navigable rivers and those used for rafting; maintenance and improvement of navigable rivers or those used for rafting and of the works which belong to them; operating and equipment; concession, construction, and operating of the railways on the quays of river ports; works of defense against rivers and torrential streams; organization and working of the syndical associations for carrying out these works; ferry services; tying-up dues, for the profit of communes, on the dependencies of the river public domain; announcement of floods; service of inspection of the ports in the basin of the Seine for the supplies of Paris.

Fishery police and fishing in canals and canalized navigable streams, not included in the dependencies of maritime fishing.

Concession and regulation of water powers and power stations and taking water from streams and canals of the public domain.

Legal questions relating to these services.

**Polders* are the low, marshy tracts of ground lying along the North Sea, which have been drained and given over to agriculture.

†*Lais* and *relais*, a tautological expression, as each word means the same thing: ground made along the side of the sea or of a stream by the sea or the stream itself.

3d Bureau. Office force: One chief of bureau, one assistant chief, one *rédacteur*, two *expéditionnaires.* Canals for navigation; construction and maintenance of canals for navigation; operating and equipment; concession, construction, and operating of railways on the quays of canal ports; supervision of canals for which concessions have been granted; navigation for pleasure; suspension of use for navigable highways; works for water supplies intended solely for canals; legal questions relating to these services.

4th Bureau. The assistant director in charge of the 2d Division has charge of this bureau. Office force: one assistant chief of bureau, one sub-engineer of the 2d class of the Ponts et Chaussées, three *expéditionnaires.* General studies *and* statistics of navigation; general questions and studies of the service of navigation; relations between railways and navigable highways or seaports; questions arising from the competition of the French inland navigable highways with other lines of transportation; centralization and coordination of statistical information regarding navigation in France; publications of the service of navigation; sundry questions concerning navigation in foreign countries; preparation and revision of the map and of the official guide for inland navigation; international congresses of navigation.

The decree of June 28, 1909, makes mention of two new classes of the inland navigation force, viz: *machinists* or *firemen* and assimilated agents, charged with the service and maintenance of machines of any kind which require special knowledge or capacity. They are divided into four classes and receive the following yearly pay:

Conductors of the 2d class: 3,000 and 2,800 francs; conductors of the 3d class: 2,600 and 2,400 francs; conductors of the 4th class: 2,200 and 2,000 francs.

Sub-engineers and conductors may also receive commutation for quarters and various other allowances laid down by the regulations. Sub-engineers and conductors are appointed by the minister, who fixes the effectives of each class in accordance with budgetary funds. Promotions from class to class require service of three years in the lower class. To obtain the higher pay in a class, a service of at least two years at the lower rate is necessary. The beginning in each class must be at the lower rate. Successive increases of pay are made by seniority, provided that no disciplinary measures have come in.

(It may be well to mention, before going further, that France is

divided, including Corsica, into 87 departments, 362 arrondissements, 2,899 cantons and 36,170 communes. At the head of each department is an administrative officer, appointed by the Department of the Interior, called the prefect, who resides at the principal town of the department. A sub-prefect, also appointed by the Department of the Interior, is at the head of the arrondissement and resides at its chief town, where is also to be found a court of first instance. The canton is simply a division of the arrondissement. The commune is presided over by a "maire" elected by the voters of the commune. The department and the commune are civil persons to receive legacies, etc. The arrondissement and canton have no such existence.

The roads of France are divided into four classes; the national roads, the department roads, the communal (or vicinal) roads and the country roads. The national roads are created, according to their importance, by a law or by a presidential decree. Funds must be provided by law. The other three classes of roads are provided for from departmental funds; they fall under the charge of the Interior Department, the Department of Public Works intervening only when requested by the Department of the Interior.)

A permanent council, called the General Council of the Ponts et Chaussées, exists in the Department of Public Works, where it takes, as a consulting body in regard to all matters concerning the Service of the Ponts et Chaussées, and in relation to the Minister of Public Works, much the same part as does the Council of State in regard to the Chief of the Executive Power. Trudaine used to bring together every Sunday, at his house, an Assembly of the Ponts et Chaussées to discuss the most important projects and the principal matters relating to the Service. This Assembly may be considered as the embryo of the present council. The decree of Fructidor 7, year XII (August 25, 1804), which reorganized the Corps of the Ponts et Chaussées, made of this almost private assembly an official council with very marked and widely extended powers. At the beginning, in spite of the extent of the territory, the slowness and difficulty of communication and the many works, it included only 5 inspectors general, who never left it, and 15 division inspectors. The Director General of the Ponts et Chaussées was the President of the Council at which were present 9 auditors of the Council of State, attached more particularly to the Administration of the Ponts et Chaussées and having a deliberative voice. There were discussed all projects of works presented,

all legal matters, all questions relating to works of art.* The opinion of this Council was transmitted by the Director General to the Minister in the form of a report, but the Minister could take no action without the Emperor's consent. The composition of the council was afterwards changed several times, especially by the decrees of June 17, 1854, of September 15, 1869, and by the decisions of May 5, 1863, and of December 19, 1878. The present composition of this council is:

The Minister of Public Works, *President*.

The Councillor of State, Director of Roads, Navigation, and Mines.†

The Director of Railways.‡

The Director of Personnel and Accounts; these three being permanent members of the Council.

The Directors of Control of railways of general interest take part in the deliberations with a right of vote in matters belonging to the direction of railways.

The other members of the Council are 11 Inspectors General of the first class, one of whom is the Vice-President, and 18 Inspectors General of the second class, of whom one is the Secretary of the Council; one, on duty with the Navy Department, has general charge of the inspection of hydraulic works, and one is director of the light-house service.

This Council holds its meetings every Thursday at 2 p. m.

The Council is divided into the four following sections:

1st Section. Has charge of roads, railways of local interest, tramways, and administrative matters relating to distributions of electric energy.

It is composed of: 2 inspectors general of the first class, one of whom is president; 5 inspectors general of the second class; 2 chief engineers of the first class as secretaries.

*The term works of art (*travaux or ouvrages d'art*) is applied in France to all works aside from the waterway proper; locks, bridges, etc., come under this head.

†There has been added to the Department of Public Works, according to the annual Year Book for 1912, which has appeared since this was written, a new direction: that of Mines, Distributions of Electric Energy and Aeronautics, of which the Director is also a member of General Council of the Ponts et Chaussées. He takes place after the Director of Personnel and Accounts.

‡According to the Year Book for 1912, the Director of Railways is also Councillor of State.

2d Section. Has charge of inland navigation; hydraulic service; fishing; communal works; questions relating to the utilization of natural motive powers.

It is composed of: 2 inspectors general of the first class, one of whom is president; 5 inspectors general of the second class; 2 chief engineers of the first class as secretaries.

3d Section. Has charge of seaports; light-house service.

It is composed of: 2 inspectors general of the first class, one of whom is president; 5 inspectors general of the second class; one ordinary engineer of the first class as secretary.

4th Section. Has charge of railways of general interest. It is composed of: 3 inspectors general of the first class, one of whom is president; 3 inspectors general of the second class, all of whom, while belonging to the General Council of the Ponts et Chaussées, are supervising directors (*directors de contrôle*), and 3 chief engineers of the first class who are also supervising directors of railways. These three chief engineers form a part of the general council only when matters relating to the railways to which they are attached have to come before the council. This section has a chief engineer of the first class as secretary.

One of the inspectors general of second class belonging to the council is its secretary, as a general rule. With him are associated at present: one chief engineer of the first class as assistant secretary; one chief engineer of the first class who prepares the minutes of the meetings of the council, and one ordinary engineer of the third class as assistant to the secretary. This last is, as a rule, the student who graduated at the head of the last class to leave the school of the Ponts et Chaussées; he holds his place for a year.

All engineers of every grade, whether on duty or on leave of absence in Paris, may be present at a meeting of the council when matters concerning their services are under discussion. The council may also call on them for information under such circumstances.

The sections act only on such matters as are not of sufficient importance to justify their being sent to the full council.

The office force of the secretary's office is composed of a chief of bureau, an assistant chief, 5 clerks of various grades and a principal draughtsman detached from the School of the Ponts et Chaussées.

The above organization of the General Council of the Ponts et Chaussées is that given by the Year Book of 1912. It differs very

materially from that found on pages 32 and 33 of Campredon's book on the Organization of the Services of Public Works in France; but it is sixteen years since that work appeared and many changes have been made.

The duties of the General Council of the Ponts et Chaussées as laid down by the decree of Fructidor 7, Year XII, were as follows: "The General Council will give its opinion on the projects and plans of works and on all questions of art and accounts which may be submitted to it and as to which a report shall have been made by those of its members who have been directed to examine them. The General Council will also give its opinion on legal questions of the administration in regard to the establishment, regulation and police of water works. It will also be consulted, necessarily, on all legal questions which are to be brought before the Council of State or to be decided by the Minister." The Council, therefore, can only *advise* or give an *opinion*, and the solution which it adopts is never binding on the administration and still less on the Council of State.

There are, in addition to the General Council of the Ponts et Chaussées, a certain number of permanent commissions or boards which sit in connection with the central administration and come directly into contact with the Ponts et Chaussées. Among the most important of these is the *Mixed Commission of Public Works*, which has to study matters connected with both the military and civil services because both are in action on the frontier zones. The origin of this institution is found as far back as 1776, in an ordinance of Turgot's, in which it is laid down: "that hereafter none of these works shall be constructed by the Administration of provinces or cities, or even by the civil engineers, unless the projects thereof shall have been communicated to the Minister of War." But the mere appreciation of engineers who were exclusively in charge of military interests did not offer a sufficient guarantee. So the law of January 19, 1791, while confirming the military servitudes of the frontier zone, created a mixed Assembly composed of members of the Corps of the Ponts et Chaussées and of the Corps of Military Engineers to advise upon the works in regard to the lines of communication of this zone. This law says: "When it is a question of works which are of interest to the roads and communications on the frontiers, and of works to be done in naval or commercial ports where the military marine is received, the projects will be discussed and examined in a mixed

Assembly composed of representatives of the Assembly of the Ponts et Chaussées and of representatives of the Corps of Engineers of the Army. The result of this examination will be laid before the Military Committee and the Committee of the Ponts et Chaussées of the National Assembly sitting together; and the legislative body will enact such legislation as shall be necessary in accordance with the report of these two committees.'' This mixed Assembly, modified under the first Empire by the decrees of June 20, 1810, and August 4, 1811, was reorganized by the ordinance of September 18, 1816. ''We are convinced,'' said the statement of reasons (*Exposé des Motifs*) of this ordinance, ''that the object of the formation of this Commission was to bring about in the examination and discussion of all projects for public works which may concern at the same time the military, civil, and naval services, a harmony of views between the various engineers attached to these three departments, so that this concerted action might lead to means of reconciliation in case of divergence of views and public interests between the various services, or offer on either side all the reasons which might cast light on the decisions to be taken by our ministers in case of dispute, and also to give, in the carrying out of any project whatever for mixed public works, the guarantee that they have been adopted under considerations which determine the true interests of the State.''

The same ordinance laid down that the Commission should be composed of:

One general officer of Engineers, member of the Committee of Fortifications;

One inspector general, member of the Council of the Ponts et Chaussées;

An inspector general of the Ponts et Chaussées, attached to the Navy Department;

Two secretaries from the Council of the Ponts et Chaussées and from the Committée of Fortifications.

This mixed Commission of Public Works was successively modified afterwards by the ordinances of December 28, 1828, of July 31, 1841, and of October 25, 1845. The law of April 7, 1851, completed by the decree of August 16, 1853, fixed the present (1896) composition finally as follows:

Four councillors of State, one of whom is president of the commission;

One inspector general of Artillery.

Two inspectors general of the Military Engineers;

One brigadier general, member of the Committee of Fortifications;

Two inspectors general of the Ponts et Chaussées;

One general officer of the Navy;

One inspector general, member of the Council of Works of the Navy.

These members are all appointed by decree and have a vote. The following are also *de jure* members of the Commission, but without a vote:

The Secretary of the General Council of the Ponts et Chaussées;

The Secretary of the Committee of the Military Engineers;

The Secretary of the Artillery Committee;

The Secretary of the Committee of Inspectors General of the Navy;

The Secretary of the Council of Works of the Navy.

The law of February 10, 1890, added:

The Secretary of the Superior Military Commission of Railways.

Finally, a lieutenant-colonel of Engineers was appointed secretary of the Commission.

The Year Book for 1912 of the Department of Public Works gives a slightly different organization from the above; it is as follows:

Four councillors of State, one of them president;

Five major generals (Généraux de division), two from the Engineers, one from the Artillery, two not mentioned;

Two inspectors general of the Ponts et Chaussées;

One rear admiral;

One inspector general of the Ponts et Chaussées, in charge of the general inspection of hydraulic works for the Navy;

One lieutenant colonel of Engineers, secretary of the Technical Committee of the Engineers;

One lieutenant colonel of Infantry, Secretary of the Superior Military Commission of Railways;

One inspector general, Secretary of the General Council of the Ponts et Chaussées;

One captain of the Navy, Secretary of the Superior Council of the Navy;

One lieutenant colonel of Engineers, *Secretary*.

The other permanent Commissions of the Central Administration which concern the Ponts et Chaussées are:

1. *The Commission for the General Levelling of France,* made up as follows:

The Minister of Public Works, *President*;

Four inspectors general of the Ponts et Chaussées, of whom one, who has charge of the ordinary service of the department of the Seine, is vice-president of the Commission, and one is a member of the Consulting Committee on Communal Roads of the Department of the Interior;

The Director of Departmental and Communal Administration in the Department of the Interior, also Vice President of the Commission;

The Director of the Geographical Service of the Army;

The Director of the Hydrographic Service of the Navy;

One retired Brigadier General, member of the Institute and of the Bureau of Longitudes, former Director of the Geographical Service of the Army;

One retired Lieutenant Colonel of Engineers;

One Colonel, Chief of the Section of Geodesy of the Geographical Service of the Army;

One Honorary Director of the Department of the Interior, former Chief of the Service of the Map of France and of Graphical Statistics;

One Member of the Consulting Committee on Communal Roads of the Department of the Interior;

One Director of Hydraulics and of Agricultural Improvements in the Department of the Interior;

One Chief Engineer of the Ponts et Chaussées, Inspector General of Agricultural Hydraulics;

One Chief Engineer of the Ponts et Chaussées, Chief of the Service of Studies of the Great Hydraulic Forces of the Region of the Alps;

One Inspector General of Mines, member of the Institute and of the Bureau of Longitudes, *Secretary of the Commission;*

One Ordinary Engineer of the Ponts et Chaussées, *Assistant Secretary.*

2. The *Commission for Announcing Floods,* made up as follows:

Seven Inspectors General of the Ponts et Chaussées, one of whom is President;

Two Chief Engineers of the Ponts et Chaussées, one of whom is the Secretary of the Permanent Commission for Inundations;

One Director of the Central Meteorological Bureau of France;

One Director of the Operating of Telegraphs;

Two Chief Engineers of the Ponts et Chaussées, *Secretary* and *Assistant Secretary* of the Commission.

3. The *Permanent Commission on Inundations*, made up as follows:

The Minister of Public Works, *President;*

Three Inspectors General of the Ponts et Chaussées, one of whom is *Vice-President;*

The Inspector General in charge of the inspection of the Services of Navigation of the Basin of the Seine;

The Inspector General in charge of the inspection of the Services of Navigation of the Basin of the Loire;

The Inspector General in charge of the inspection of the Services of Navigation of the Basin of the Rhône;

The Inspector General in charge of the inspection of the Services of Navigation of the Basin of the Garonne;

One Chief Engineer of the Ponts et Chaussées, Chief of the Technical Service of Water and Drainage of the Municipal Service of the City of Paris;

One Director of the Central Meteorological Bureau;

Two Representatives of the Department of Agriculture;

One Representative of the Department of the Interior;

Two Chief Engineers of the Ponts et Chaussées, of whom one is the *Secretary* of the Commission.

SERVICE OF GAUGING RIVERS.

This service belongs to the Permanent Commission on Inundations. It is in charge of—

One Director, Inspector General of the Ponts et Chaussées, who is President of the Commission for announcing floods, as well as Vice-President of the Commission on Inundations;

Two Chief Engineers of the Ponts et Chaussées, who are the Secretary and Assistant Secretary of the Commission for announcing floods.

4. The *Commission on National Roads*, made up as follows:

Eight Inspectors General of the Ponts et Chaussées, of whom one is President;

One Chief Engineer of the Ponts et Chaussées, Secretary of the

second Section of the General Council of the Ponts et Chaussées. Secretary.

5. The *Commission on Inventions,* composed of:

The Director of the National School of the Ponts et Chaussées, President;

The Inspector and the Professors of the School.

6. *Commission on Fishing in Rivers,* composed of:

One Councillor of State in ordinary service, *President.*

Members as Representatives.

1 *Of the Department of Agriculture*—

Two Administrators of Waters and Forests;

One Inspector of Waters and Forests;

One Assistant Inspector of Waters and Forests.

2. *Of the Department of Public Works*—

Three Inspectors General of the Ponts et Chaussées;

One *Rédacteur* from the Sub-Direction of Navigation.

7. *Commission of Light-houses,* composed of:

The Minister of Public Works, President;

One Vice-Admiral, Vice-President;

One Inspector General of Mines, Member of the Academy of Sciences;

Two Rear Admirals;

One Inspector General of Maritime Engineers;

One Director of Hydrography in the Navy Department;

One Inspector General of the Ponts et Chaussées, in charge of the general inspection of naval hydraulic works;

One Inspector General of the Ponts et Chaussées, in charge of the Direction of the Service of Light-houses and Beacons;

Three Inspectors General of the Ponts et Chaussées;

One Chief Engineer of the Ponts et Chaussées, in charge of the Central Service of Light-houses and Beacons.

The Inspector General in charge of the Direction and the Chief Engineer in charge of the Central Service act as Secretary and Assistant Secretary.

One Ordinary Engineer of the Ponts et Chaussées, attached to the Central Service, is present at and may take part in the deliberations of the Commission, but without vote.

8. *Commission of Limes and Cements,* composed of:

Four Inspectors General of the Ponts et Chaussées, of whom one is President;

One Inspector General of Mines;

Two Chief Engineers of the Ponts et Chaussées;

The Chief of the Laboratory of the Ponts et Chaussées at Boulogne-sur-mer;

The former President of the Syndical Chamber of Limes and Cements;

The President of the Syndical Chamber of the Manufacturers of Artificial Portland Cement of France;

The Engineer Director of the General Society of the French Cements;

One Ordinary Engineer of the Ponts et Chaussées, Secretary.

One Conductor of the Ponts et Chaussées, attached to the office of the Secretary.

9. *Commission of Distribution of Electrical Energy,* composed of:

De jure members.

The Director of Roads and Navigation, or his delegate;

The Director of Railways, or his delegate;

The Director of Personnel and Accounts, or his delegate;

The Director of Mines, Distributions of Electrical Energy and Aeronautics, or his delegate;

The Representatives of the Department of Public Works, Posts, and Telegraphs on the Permanent Committee of Electricity;

The Director of Departmental and Communal Administration of the Department of the Interior;

The Inspector General of the Services of Control of Distributions of Electric Energy.

Members Appointed by the Minister.

Nine Inspectors General of the Ponts et Chaussées, of whom one is President;

One Inspector General of Mines;

One Chief Engineer of Mines;

One Departmental Inspector of Labor;

The General Manager of the Metropolitan Railway Company of Paris;

The Director of the Society, "Electrical Energy of the Shore of the Mediterranean;"

The Director of the Society, "Triphasé;"

The General Manager of the Society of the Motive Power of the Rhône at Lyons.

Office of the Secretary of the Commission.

One Ordinary Engineer of the Ponts et Chaussées, Secretary pro tem.

Three Ordinary Engineers of the Ponts et Chaussées, One Engineer of Telegraphs, Assistant Secretaries and Reporters.

One *Rédacteur* and *One Expéditionnaire* attached to the office.

10. *Consulting Committee for Settling Out of Court Contracts for Public Works and for Furnishing Supplies.*

One Retired Inspector General of the Ponts et Chaussées, President;

One Councillor of State;

Two Inspectors General of the Ponts et Chaussées;

One Civil Engineer;

One Chief Engineer of the Ponts et Chaussées, Assistant Reporter;

One Secretary;

One Sub-Engineer, attached to the Committee.

11. *Commission Charged With Presenting Propositions in Regard to Engineers and Conductors of the Ponts et Chaussées Residing Abroad.*

Three Inspectors General of the Ponts et Chaussées, of whom one is President.

12. *Council of Inquiry.*

Members taking part in all deliberations:

One Inspector General of the Ponts et Chaussées, President;

One Chief Engineer of the Ponts et Chaussées;

One Ordinary Engineer of the Ponts et Chaussées.

Members taking part in the deliberations only when functionaries or agents of their own class are in question:

Two Sub-Engineers of the Ponts et Chaussées;

Two Sub-Engineers of Mines;

Two Controllers of the Work of Railway Agents;

Two Commissaries of Administrative Oversight of Railways;

One Principal Clerk of the Ponts et Chaussées;

One Clerk of the Ponts et Chaussées;

Two Port Masters;

One Guardian of Navigation;

One Head Lock-keeper;

Four *Rédacteurs* in the Secretary's office.

13. *Commission for the Study of Simplification of Administrative Machinery and the Revision of Methods.*

This Commission is composed of:

Two Inspectors General of the Ponts et Chaussées, one as President, the other as Vice-President;

One Secretary General, unattached, private secretary to the Minister;

One Sub-Director of Personnel and Accounts;

Two Chief Engineers of the Ponts et Chaussées;

One Chief Engineer of Mines;

One Ordinary Engineer of the Ponts et Chaussées;

One Sub-Engineer of the Ponts et Chaussées;

One Principal Clerk of the Ponts et Chaussées;

One Sub-Engineer of Mines;

One Principal Inspector of the Commercial Operating of Railways;

One Principal Conductor of the Ponts et Chaussées, Secretary.

14. *Commission of the Annales des Ponts et Chaussées.*

This Commission is composed of:

The Director of Personnel and Accounts;

The Councillor of State, Director of Roads and Navigation;

The Councillor of State, Director of Railways;

The Director of Mines, Distributions of Electric Energy and Aeronautics;

The Inspectors General of the First Class of the Ponts et Chaussées;

The Secretary of the General Council of the Ponts et Chaussées;

The Engineers of the Ponts et Chaussées who are Professors at the National School of the Ponts et Chaussées;

The Director of the School is the President of this Commission, of which the Inspector of the School is the Secretary;

Two Chief Engineers of the first class are Assistant Secretaries.

In addition to the above commissions, there are in the Department of Public Works several others which do not touch especially the Ponts et Chaussées; such are:

The *Superior Military Commission of Railways,* which out of 25 members has but one representative, an Inspector General, of the Ponts et Chaussées;

The *Consulting Committee of Railways,* which out of about 100 members has but seven representatives. From the way in which

the membership of this Committee is stated in the Year Book, it is impossible to give the exact number of its members.

The *Committee on the Technical Operating of Railways,* which out of about 90 members has but 15 representatives of the Ponts et Chaussées;

The *Commission for the Verification of the Accounts of the Railway Companies,* which has about half of its members from the Ponts et Chaussées;

The *Military Commission of Navigation* which has four members, two engineers of the Ponts et Chaussées and two army officers;

The *Military Commission of Mines* has no representative from the Ponts et Chaussées;

The *Permanent Commission of Scientific Research on Fire Damp and the Explosives Used in Mines* has no representative;

The *Committee on Electricity,* with 30 members, has three representatives of the Ponts et Chaussées, one of them being President;

The *Central Commission on Steam Engines,* with 25 members, has two representatives of the Ponts et Chaussées, one of them being Vice-President;

The *Central Commission on Automobiles,* with 32 members, has six representatives, of whom are the President, Vice-President, one of the Reporters and one of the Secretaries;

The *Special Commission of the Geological Map of France and of the Geological Map of Algeria* has no representative of the Ponts et Chaussées;

The *Commission to Examine and Coordinate Statistical Information Regarding Mineral Industry and Steam Engines* has no representative of the Ponts et Chaussées;

The *Commission on Aerial Navigation* has three representatives of the Ponts et Chaussées, of whom one is Vice-President; one of the assistant secretaries belongs to this corps;

The *Commission of the Annales des Mines* is composed exclusively of Engineers of Mines and Professors at the School of Mines.

Such, then, is the Central Administration, which has for its object to centralize all matters belonging to the Service of the Ponts et Chaussées, to pass on expenditures and to approve, finally, all projects.

(*To be continued.*)

Editorial Notes

Coming Articles

The Editors of the MEMOIRS are pleased to announce that they have a number of good articles on hand or promised for coming issues, among the most important of which is one on "Tidal Currents in East River, New York," by Col. W. M. Black. This will be accompanied by about fifteen plates, and on the whole is a very valuable and exhaustive treatise on the currents and the conditions caused by them in the East River, New York City, particularly around Hell Gate.

Another article received is on the Dalles Celilo Canal, Columbia River, which is being built by the Government at a cost of about $4,500,000 largely by hired labor. This article deals historically of early conditions on the upper Columbia and Snake rivers, as well as technically with constructional details, and the difficulties encountered during construction.

Another excellent article promised is on the Colbert Shoals Canal, Tennessee River, while a shorter but valuable one on drilling operations on the Tuscumbia Bar, Tennessee River, has already been received.

Along with these we have a number of other articles of value and interest to both civil and military engineers.

Water Power

In January of this year there appeared the first issue of a new magazine, entitled "Water Power," under the management of a well-known engineer. The writer welcomes this addition to the literature on the National resources of the United States, for he believes that the subject of "Water Power," is one of sufficient importance and wide general interest to support a separate publication. On the other hand, there are springing up to-day so many publications financed or backed by corporations and moneyed interests, particularly interested in the subject treated in the magazine, that one is inclined to suspend judgment upon a publication until time shall indicate its policy.

While we have heard much on this subject of "Water Power" in the past three or four years, the indications are that we will

hear a great deal more in the next three or four years. At present the discussions seem to be divided into two general classes: those favoring National control and those favoring State control. This is not peculiar to Water Power alone, but applies as well to forests, to coal, iron, copper, petroleum, and other National resources. The writer believes that all of these resources belong to the nation and should be administered for the benefit of the whole people, and that this can only be done by the National Government. To place them under the control of the State governments will make just that much easier their passage into the hands of corporations seeking to control them for their own private benefit. This is brought about by raising the local cry that the administration of these resources as carried out by the National Government through leases and other means is retarding the development of the State or locality and withholding from the residents of that locality the wealth that properly belongs to them.

If the publication, "Water Power," above referred to shall prove to be one dealing fairly and broadly with the question of Water Power and the public's rights therein, it will be welcomed; if it should prove to be otherwise, it will readily pass to the waste basket along with numerous other publications designed solely to advance private interests of one sort or another. We sincerely trust that it shall prove to be the former.—A. A. F.

Errata

The following table of costs is furnished by Maj. E. H. Schulz to replace table of cost shown on pages 710 and 713, PROFESSIONAL MEMOIRS, No. 18.

a. Willow Mattress.		
757 cords brush, at $0.892	$675.87	
776 cubic yards ballast, at $0.746	579.09	
555 cubic yards spalls, at $0.702	389.70	
Wire strand, clips	204.43	
Towboat service	784.00	
Labor, superintendence and miscellaneous	2,669.59	
		$5,302.68
b. Concrete Paving.		
1,156 cubic yards gravel, at $1.533 plus	$1,772.35	
Grading bank, 1,000 linear feet, at $0.94607	946.07	
Reinforcement	749.32	
821¾ barrels Portland cement, at $1.15	945.01	
Labor, superintendence, and miscellaneous	1,881.33	
		6,294.08
c. Flexible Concrete Blocks.		
120 cubic yards gravel, at $1.533	$183.96	
Reinforcement	221.65	
182 barrels Portland cement, at $1.15	209.30	
Labor, etc., making blocks	636.00	
Labor, etc., placing blocks	685.12	
		1,936.03
Summary.		
Weaving and ballasting mattress, 1,030 linear feet at $5.1482		$5,302.68
Concrete paving and grading, 1,000 linear feet, at $6.294		6,294.08
Flexible concrete blocks in place		1,936.03
Total		$13,532.79

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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Contents

Illustrations

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BVT. BRIG. GEN. SIMON BERNARD

CORPS OF ENGINEERS, UNITED STATES ARMY

1816-1831

BORN 1779—DIED 1839

SEE P. 306

Tidal Movements in East River, New York*

BY

Col. W. M. Black
Corps of Engineers; Member American Society Civil Engineers

REDUCTION OF CURRENT VELOCITIES.

An inspection of the directions and velocities of the East River surface currents, as given by the United States Coast and Geodetic Survey, shows how the velocities are affected by the channel sections. The removal of the Corlears Hook Reef and the opening of the east channel at Blackwells Island will tend to diminish the high current velocities in their respective reaches. The records show that from January 1, 1908, to January 31, 1912, there were thirty-eight reported accidents in the East River, *exclusive of the Hell Gate reach,* ranging in extent from the loss of a vessel to minor injuries, almost all of which were due to the current velocities.

The worst conditions are found in Hell Gate. Here at about two hours after the low water of a maximum spring tide at Hallets Point, the surface currents are northerly and attain a maximum velocity past Negro Bluff of 6.7 miles per hour. Currents to the south attain a maximum surface velocity of 6.35 miles per hour and about 2½ hours after a spring tide high water. For tides of mean height these surface velocities are about 6.1 miles and 5.5 miles per hour, respectively. Greater velocities occur over the reef off Hallets Point. It is easy to understand that with such velocities low-powered boats can not stem the current and that all boats are to a certain extent unmanageable. Collisions with the rocks and with other vessels are of frequent occurrence. Tows are compelled to await slack water or a favoring current, and many are afraid to risk the passage at night.

In the period from January 1, 1908, to January 31, 1912, inclusive, there is a record of thirty-five more or less serious disasters

*Extracts from a report on the East River, submitted by Colonel Black.

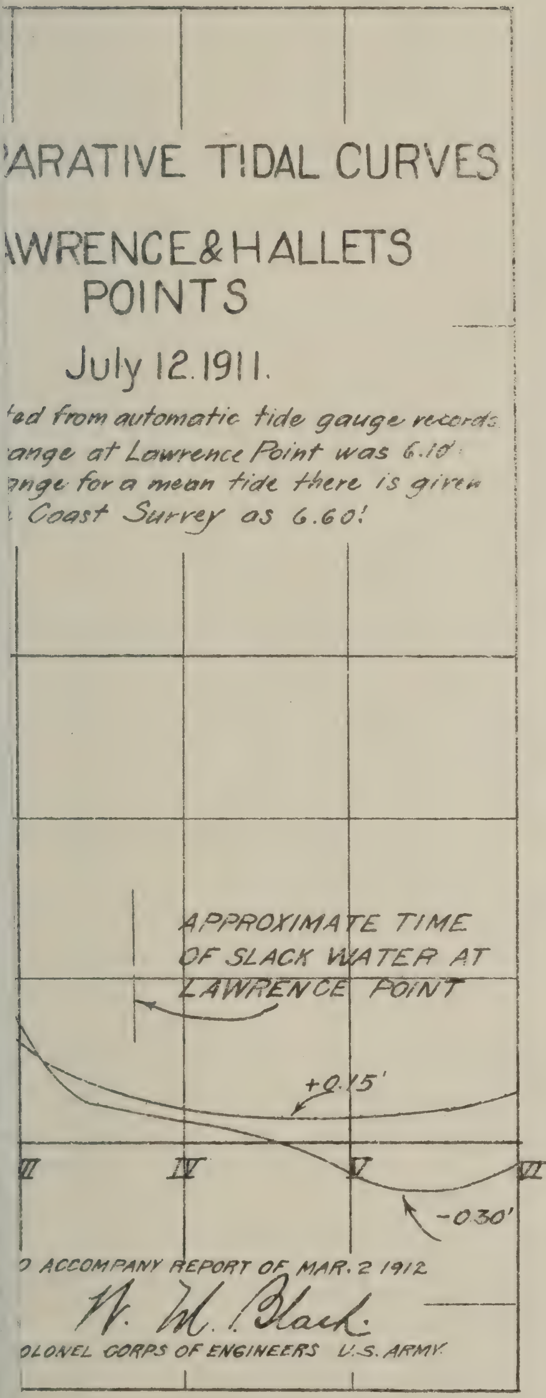
to vessels in Hell Gate. Undoubtedly minor disasters in addition occurred which were not reported.

To understand the causes of these velocities a short discussion of the tidal phenomena is necessary. The tides at Hell Gate are due to the tidal waves entering the harbor through the Sandy Hook and Long Island Sound entrances. The wave coming from the Atlantic is retarded in its passage through Long Island Sound by four hours, that is, the time of high water at Sandy Hook is four hours earlier than the time of high water at Throgs Neck. The wave arriving at Sandy Hook is transmitted through the lower and upper bays and thence to the East River at Hell Gate by two routes, one direct through the East River, the other via the Hudson and Harlem rivers. Similarly, the wave entering at Throgs Neck from Long Island Sound is transmitted via the East River and via the Hudson and Harlem rivers to the New York upper bay. The tidal curves south of High Bridge in the Harlem and those in Hell Gate show by inflections the superposition of the two waves.

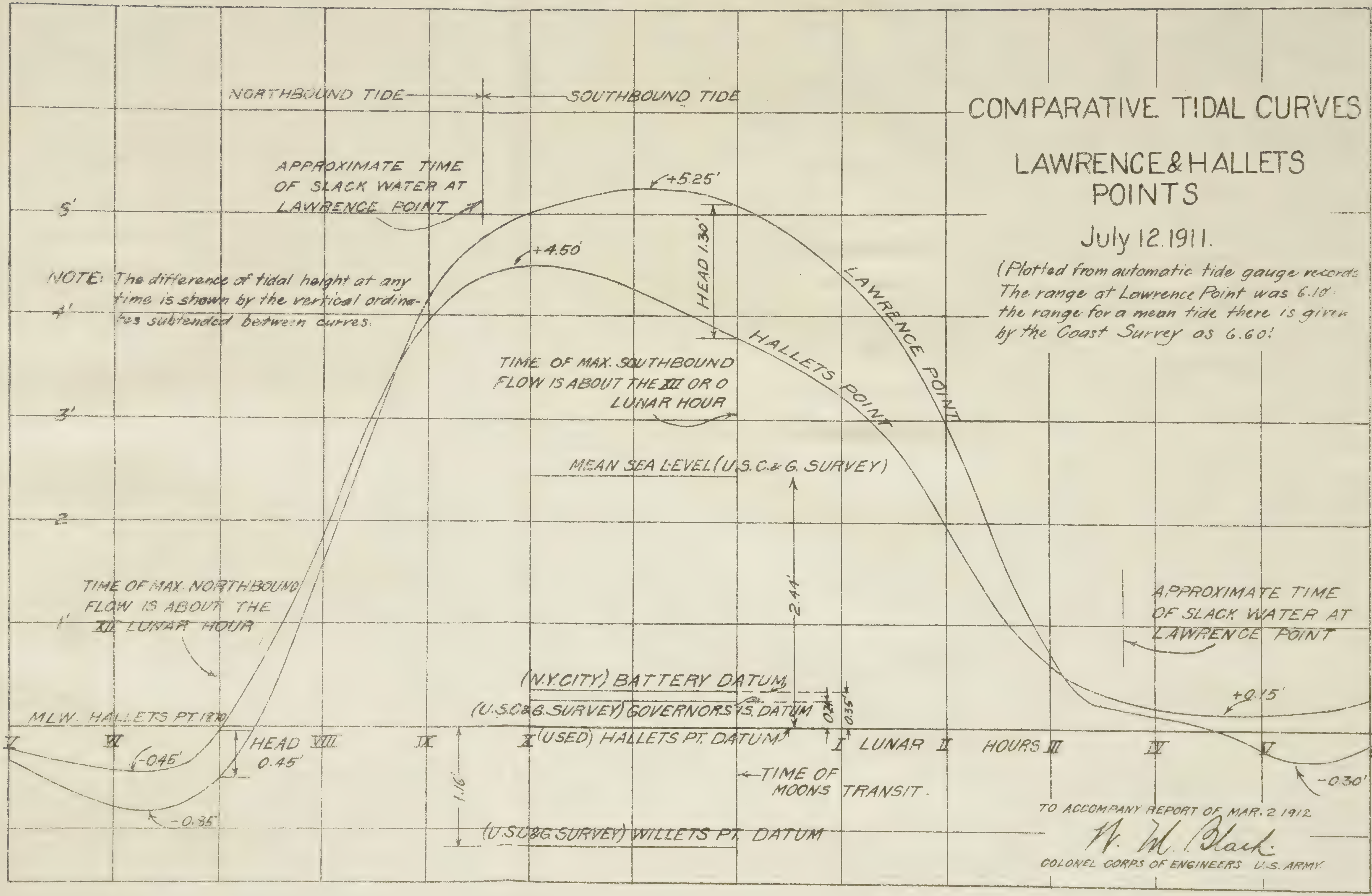
The current velocities are caused by the local slopes produced by these waves. These currents are an actual movement of the waters horizontally, while the wave movement is a vertical motion shown by an increasing or decreasing depth at a given point. It is evident that the waters moving horizontally acquire a certain living force and that, if an impediment to the movement be met, this force will be expended in a vertical movement, causing an increase of depth. If the impediment be impassable, the water surface will continue to rise until the living force is exhausted. If the impediment be a simple narrowing of a channel, the rise will be continued until the increase of slope toward the outlet is great enough to increase the velocities of the outflowing current to a point sufficient to afford the requisite discharge.

This fact is well illustrated by a comparison of the tidal curves at Lawrence Point and at Hallets Point (Plate 2 opposite). About three or three and one-half hours after the moon's transit, the currents being southerly, the water surface at Lawrence Point becomes actually lower than the surface at Hallets Point, but the southerly flow continues for several minutes before the living force of the flowing water is exhausted, by which time the surface at Hallets Point has become about 0.1 foot higher than the surface at Lawrence Point. Then a change of current direction takes place throughout the lower East River, following a change which has

Plate 2



'ARATIVE TIDAL CURVES
\WRENCE & HALLETS
POINTS
July 12.1911.
'ted from automatic tide gauge records.
'ange at Lawrence Point was 6.10'.
'ange for a mean tide there is given
. Coast Survey as 6.60'.
APPROXIMATE TIME
OF SLACK WATER AT
LAWRENCE POINT
+0.15'
III
IV
V
VI
-0.30'
'O ACCOMPANY REPORT OF MAR. 2 1912
W. M. Black.
'OLONEL CORPS OF ENGINEERS U.S. ARMY.


COMPARATIVE TIDAL CURVES
LAWRENCE & HALLETS POINTS
July 12. 1911.
(Plotted from automatic tide gauge records. The range at Lawrence Point was 6.10: the range for a mean tide there is given by the Coast Survey as 6.60'.)
NORTHBOUND TIDE
SOUTHBOUND TIDE
APPROXIMATE TIME OF SLACK WATER AT LAWRENCE POINT
+5.25'
+4.50'
HEAD 1.30'
LAWRENCE POINT
HALLETS POINT
NOTE: The difference of tidal height at any time is shown by the vertical ordinates subtended between curves.
TIME OF MAX. SOUTHBOUND FLOW IS ABOUT THE XII OR O LUNAR HOUR
MEAN SEA LEVEL (U.S.C.& G. SURVEY)
TIME OF MAX. NORTHBOUND FLOW IS ABOUT THE XII LUNAR HOUR
APPROXIMATE TIME OF SLACK WATER AT LAWRENCE POINT
(N.Y.CITY) BATTERY DATUM
(U.S.C.& G.SURVEY) GOVERNORS IS. DATUM
MLW. HALLETS PT. 1870
(USED) HALLETS PT. DATUM
2.44'
0.35'
+0.15'
HEAD 0.45'
-0.45'
-0.85'
-0.30'
1.16'
V
VI
VIII
IX
X
I
LUNAR
II
HOURS
III
IV
V
VI
TIME OF MOONS TRANSIT.
(U.S.C.& G.SURVEY) WILLETS PT. DATUM
5'
4'
3'
2
1'
TO ACCOMPANY REPORT OF MAR. 2 1912.
W. M. Black
COLONEL CORPS OF ENGINEERS U.S. ARMY.

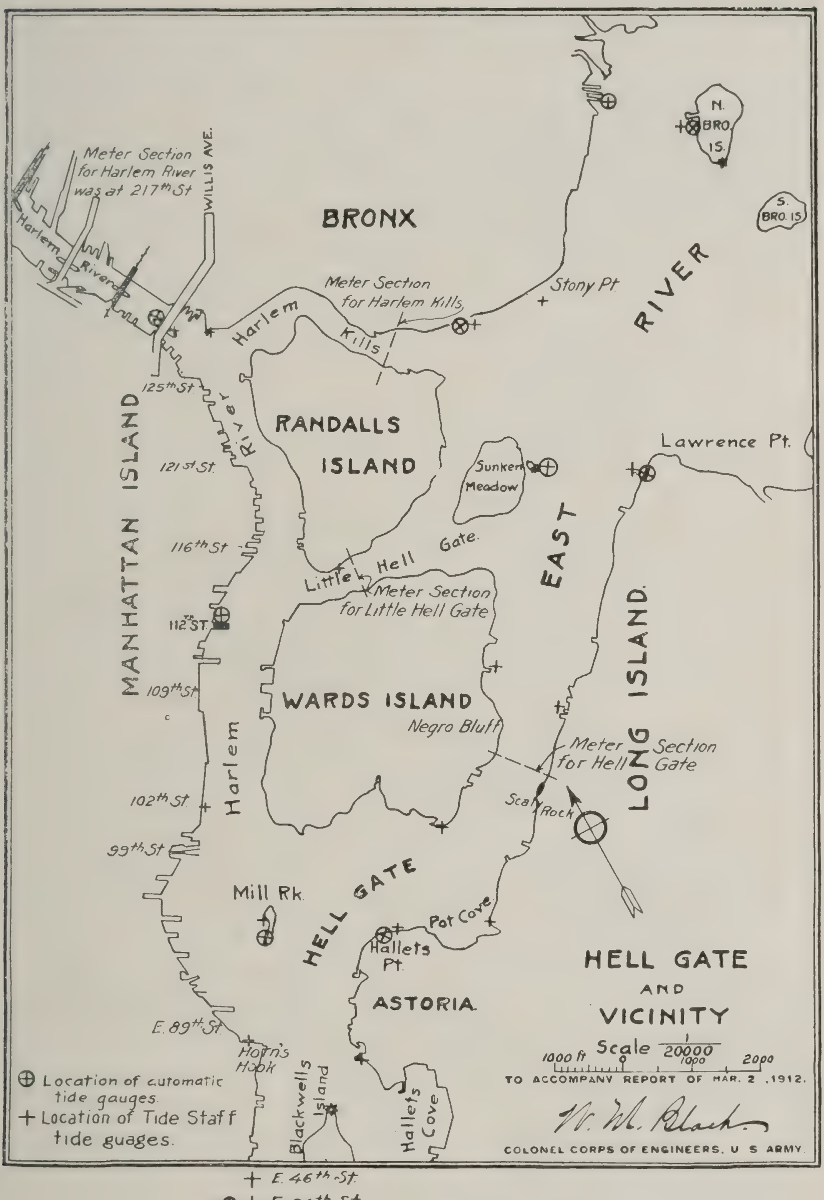


Plate 1. (See bottom page 256.)

already taken place east of North Brother Island, the current direction now being toward the Sound. But the water is engorged between Hallets Point and Lawrence Point. The fall of surface is more rapid at Lawrence Point than at Hallets Point, and the current velocities are increased until they become a maximum at about seven lunar hours after the moon's transit, when the difference of level is about one-half foot, after which time the difference of level and the velocities gradually decrease.

About eight and one-half or nine lunar hours after the moon's transit (the tide still flowing northward) the water surface level at Lawrence Point becomes higher than at Hallets Point, but the northerly currents continue for perhaps half an hour longer and until the Lawrence Point level is about 0.40 foot higher than that at Hallets Point, when slack water and a change of current direction takes place throughout the East River, following again a change of direction east of North Brother Island, the current direction now being southerly or toward Sandy Hook. But again the effects of the engorgement of the channel are seen. The water flows away from Hallets Point more rapidly than it is supplied. The surface at Lawrence Point continues to rise. A head of more than 1 foot is produced. Maximum currents are again generated, gradually decreasing as the difference in head diminishes. Harlem Kills and Little Hell Gate have their maximum discharge at these high water periods.

The dotted lines on Plate 6B, near the points of high and low water, show what would be the shape of the tidal curve in the case of Little Hell Gate were the velocities those actually due to the difference of level, or head, between the two points at that time. The difference between the full curve and its dotted curve in each case is a measure of the effect of the living force of the flowing water.

A comparison of the tidal curves for Mill Rock in Hell Gate. Willis Avenue in the lower reach of the Harlem, and Hallets Point, showed that the Mill Rock and Willis Avenue curves were practically coincident and followed fairly closely the curve for Hallets Point. Comparing these curves with the Lawrence Point curve, it is seen that at high and low water at Lawrence Point, about the sixth and eleventh lunar hours, there is a difference of level of 0.7 foot and of 1.2 feet, respectively, between the levels at Mill Rock and the lower Harlem River on the one hand and the level of the East River just beyond Hell Gate on the other.

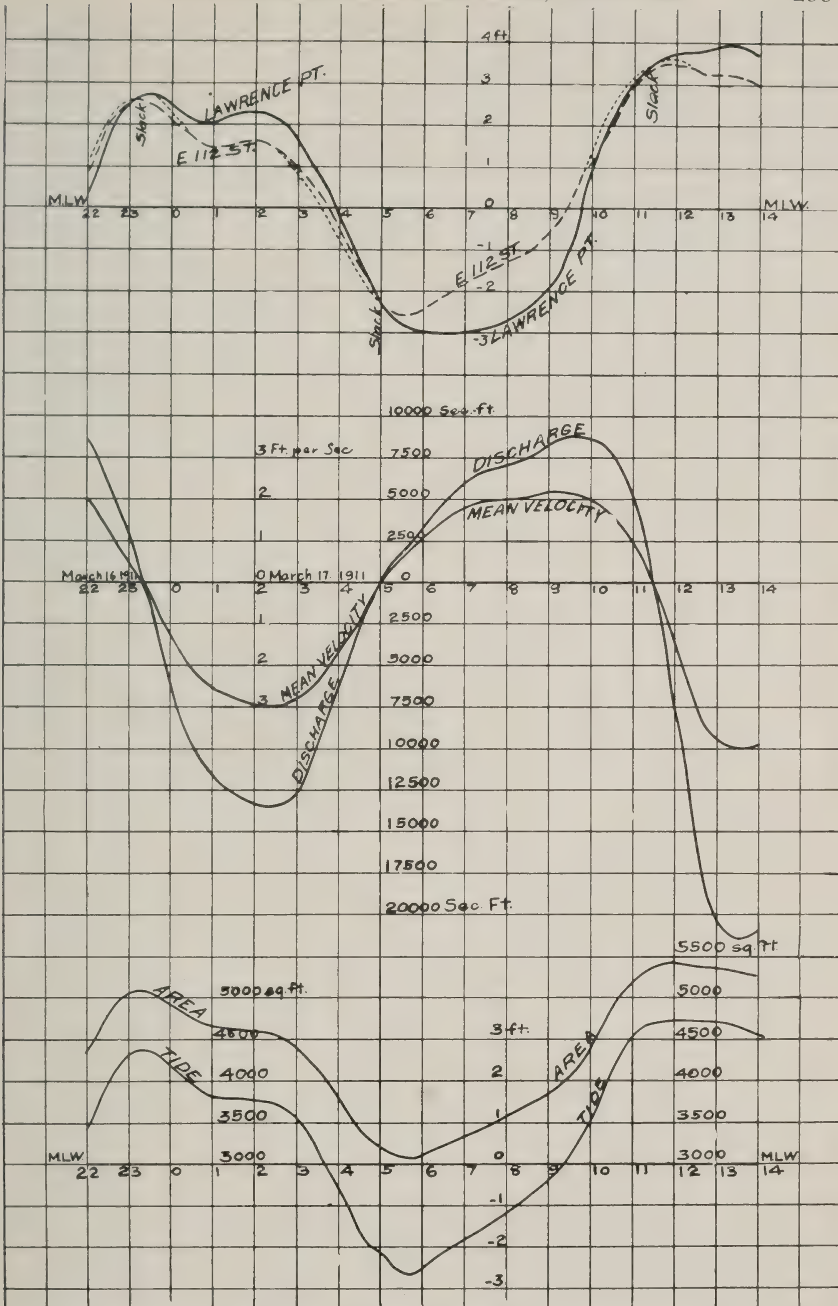


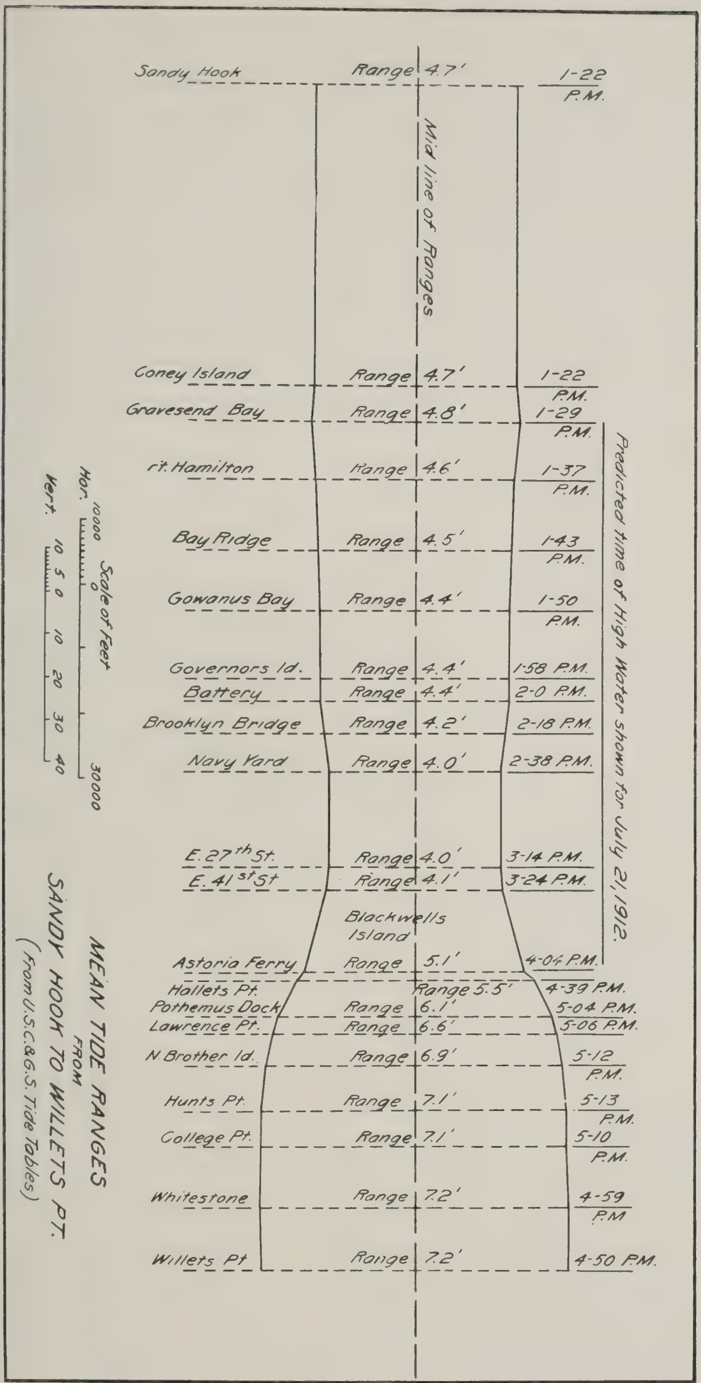
Plate 6B. Discharge sheet, Little Hell Gate. Comparative tidal curves. Note: Difference of level (head) at any time shown by vertical ordinate subtended between the tidal curves. Effective head near slack measured between Lawrence Point curve and dotted curve.

The reaches of the lower Harlem River and of the East River through Hell Gate are connected: (1) By the direct channel lying between Wards Island and the Long Island shore; (2) by Little Hell Gate between Wards Island and Randalls Island; and (3) by the Harlem (Bronx) Kills between Randalls Island and the Borough of the Bronx. Both the Little Hell Gate and the Harlem Kills channels are badly obstructed by rocks, so that there is a marked difference in their discharge capacities at high water and at low water. At low water the flow is northerly, or from the lower East River and the Harlem toward the Sound. At high water the flow is southerly, or from the Sound toward the lower East River and the Harlem. The influence of the diminished cross section and diminished discharge through the minor channels is shown by the greater differences of head produced on the two sides at low water than at high.

As might be inferred from the tidal slopes (Plate 16B, two sheets opposite page 256), there is but slight circulation in the Harlem River between the Harlem Kills and the south end of Wards Island.

Plate 16B (in two sheets) shows the slopes of the water surface for a northbound tide at the seventh lunar hour, between 24th Street, East River, and North Brother Island. These slopes are not strictly accurate, since the readings had to be taken at the tide gauge stations, as marked on Plate 1 of this report, and breaks probably occur at points other than directly at the stations. The sharp break shown beyond 89th Street marks the effect of the obstructions at Hell Gate between the north end of Blackwells Island and Lawrence Point. As will be seen, the velocities produced by these steep slopes are sufficient to cause the water to flow with an upward slope for some little distance beyond Lawrence Point.

Plate 3 shows the the effect of this obstructed passage on the tidal waves themselves. In this plate the tidal ranges at various points along the East River have been plotted with their middle points on a straight line with the channel lengths between them to scale. Between Willets Point and North Brother Island the range and velocity of transmission are due almost entirely to the wave entering through Long Island Sound. Between Sandy Hook and the Battery, it is the wave entering from Sandy Hook which is recorded. The intermediate ranges between the Battery and North Brother Island are more or less affected by the superposition and interference of the two tidal waves, the meeting place being given as Hell Gate by the United States Coast and Geodetic Survey. It



Sandy Hook Range 4.7′ 1-22 P.M.
Mid line of Ranges
Coney Island Range 4.7′ 1-22 P.M.
Gravesend Bay Range 4.8′ 1-29 P.M.
Ft. Hamilton Range 4.6′ 1-37 P.M.
Bay Ridge Range 4.5′ 1-43 P.M.
Gowanus Bay Range 4.4′ 1-50 P.M.
Governors Id. Range 4.4′ 1-58 P.M.
Battery Range 4.4′ 2-0 P.M.
Brooklyn Bridge Range 4.2′ 2-18 P.M.
Navy Yard Range 4.0′ 2-38 P.M.
E. 27th St. Range 4.0′ 3-14 P.M.
E. 41st St Range 4.1′ 3-24 P.M.
Blackwells Island
Astoria Ferry Range 5.1′ 4-04 P.M.
Hallets Pt. Range 5.5′ 4-39 P.M.
Pothemus Dock Range 6.1′ 5-04 P.M.
Lawrence Pt. Range 6.6′ 5-06 P.M.
N. Brother Id. Range 6.9′ 5-12 P.M.
Hunts Pt. Range 7.1′ 5-13 P.M.
College Pt. Range 7.1′ 5-10 P.M.
Whitestone Range 7.2′ 4-59 P.M.
Willets Pt Range 7.2′ 4-50 P.M.
Predicted time of High Water shown for July 21, 1912.
Scale of Feet
Hor. 10000 0 30000
Vert. 10 5 0 10 20 30 40
MEAN TIDE RANGES
FROM
SANDY HOOK TO WILLETS PT.
(From U.S.C.&G.S. Tide Tables)

Plate 3.

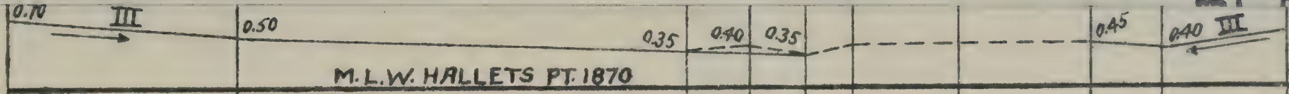
will be noted, that as the tidal wave enters from the Sound the rate of transmission and the range diminish gradually until North Brother Island is reached. From that point south, the range and rate of transmission diminish at first gradually and then rapidly as far as Astoria Ferry; then comes a more gradual diminution of range and a slight increase of rate as far as East 27th Street, between which point and the Navy Yard there is a reach in which the range is uniform. From the Navy Yard to the sea through the Sandy Hook entrance the tidal ranges and rates of transmission gradually increase as the channels widen and deepen. The abrupt change of range and of rate of wave transmission between North Brother Island and Astoria Ferry are undoubtedly due to the obstructed channels of Hell Gate.

Were the channel made uniform in size between North Brother Island and East 41st St., these abrupt breaks would be eliminated. Such a channel would probably have a practically uniform surface slope throughout the entire reach at any moment with a uniformly varying velocity of wave transmission and with current velocities less than those which now are found in portions of the obstructed reach. In other words, in such a uniform channel, tidal ranges and wave and current velocities would become more nearly uniform. It would seem to be a logical deduction that improvements of channel dimensions in this reach in the direction of uniformity will tend toward these conditions—that the tidal range will be made more uniform and the excessive slopes and resultant current velocities diminished. Similar changes in tidal range have been marked in tidal rivers after their channels have been improved.

The deepening of the channel east of Blackwells Island and the opening of the obstructed passages to the north and south of Randalls Island (Harlem Kills and Little Hell Gate) would be improvements in the direction of such uniformity.

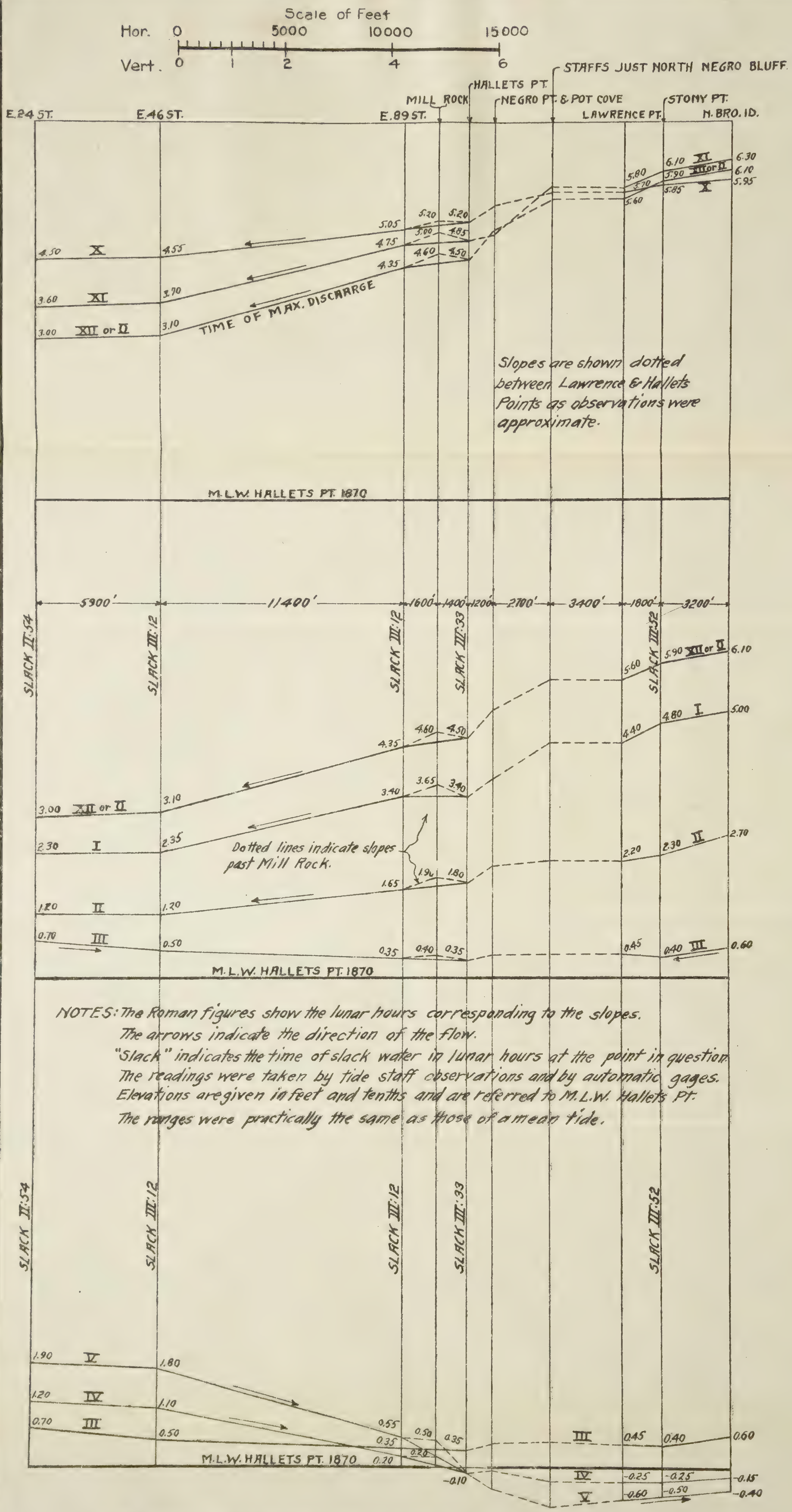
TIDAL CIRCULATION IN THE VICINITY OF HELL GATE.

The tidal flow through each of the Hell Gate passages is closely interrelated. On the northbound, or flood tide, water is flowing north in the East River and south in the Harlem River. The combined discharge from these two rivers flows on out the East River towards Long Island Sound. That which comes up the East River passes mostly through Hell Gate, but a small portion may pass up the Lower Harlem and through Little Hell Gate. The Harlem discharges principally through Harlem Kills into the East River, and a portion through Little Hell Gate (Plate 1, see page 251).



NOTES: The Roman figures show the lunar hours corresponding to the slopes.
The arrows indicate the direction of the flow.
"Slack" indicates the time of slack water in lunar hours at the point in question.
The readings were taken by tide staff observations and by automatic gages.
Elevations are given in feet and tenths and are referred to M.L.W. Hallets Pt.
The ranges were practically the same as those of a mean tide.

PLATE-16 B

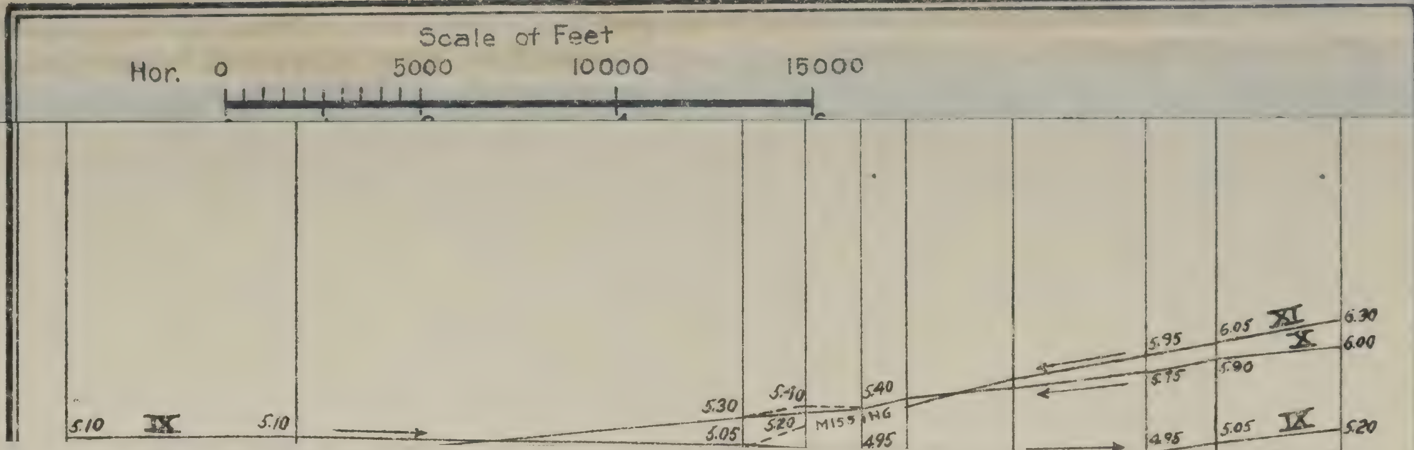


SOUTHBOUND TIDE, OBSERVED JULY 20, 1912.

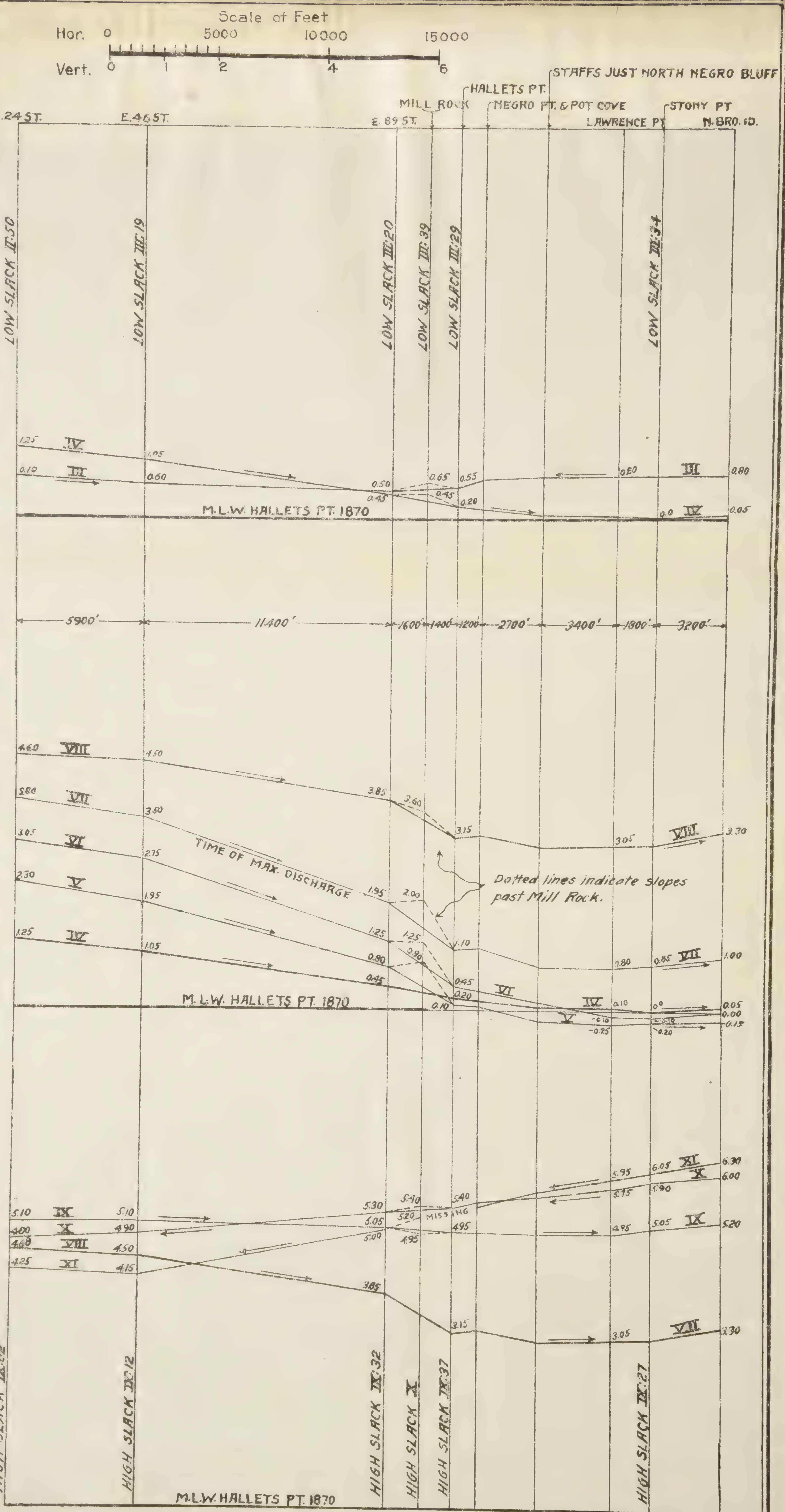
THIS AND FOLLOWING SHEET SHOW

COMPARATIVE TIDAL SLOPES

E. 24TH ST. THROUGH HELL GATE TO N. BROTHER ID.



Scale of Feet
Hor. 0 5000 10000 15000
5.10
IX
5.10
5.30
5.05
5.40
5.20
MISSING
5.40
4.95
5.95
5.75
4.95
6.05
5.90
5.05
XI
X
IX
6.30
6.00
5.20



NORTHBOUND TIDE, OBSERVED JULY 21, 1912.

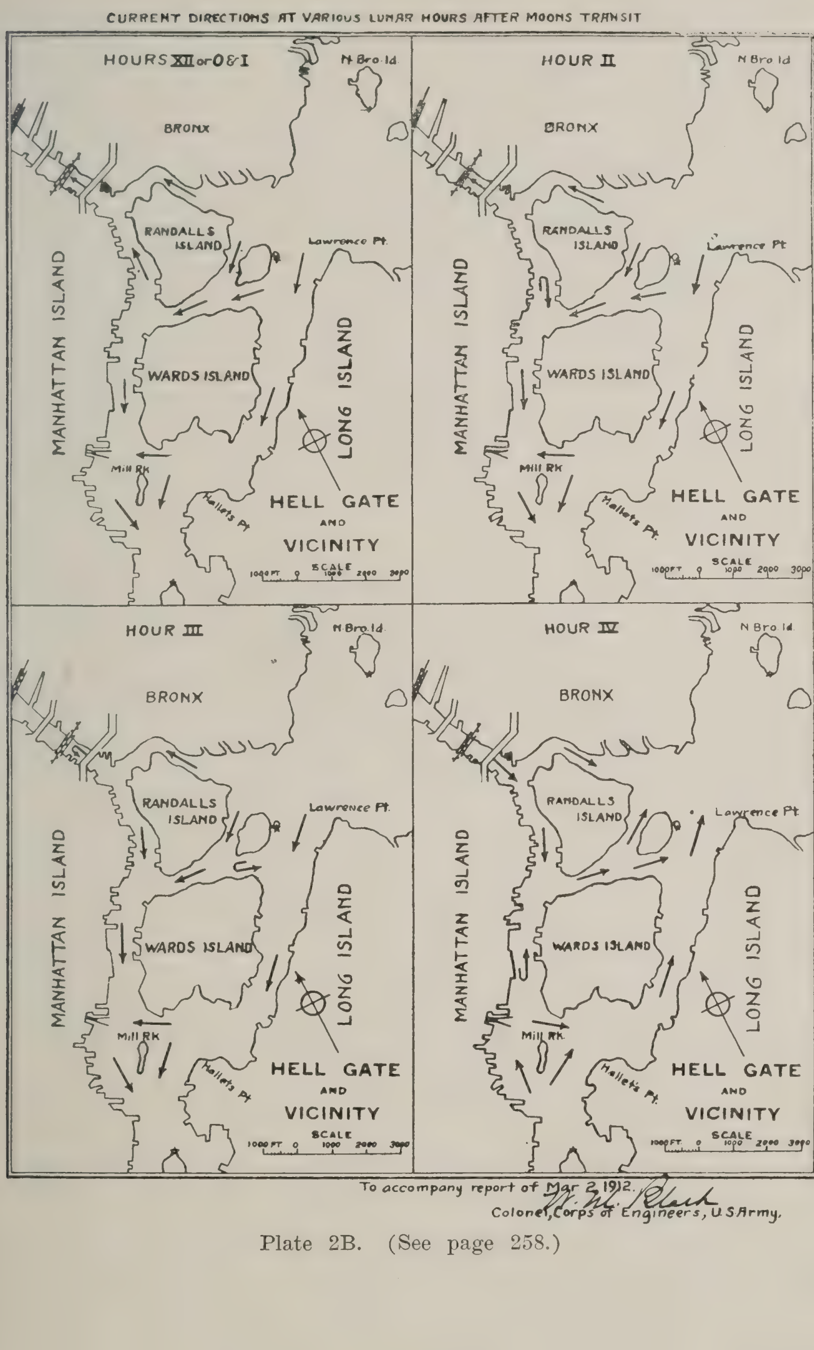


Plate 2B. (See page 258.)

Of the flow through Little Hell Gate on the northbound tide, the greater portion passes into the East River through the channel between Sunken Meadow and Randalls Island. There is also a fairly strong flow eastward through Little Hell Gate *east*, between Sunken Meadow and Wards Island. On the southbound tide, however, Little Hell Gate is fed entirely through the Sunken Meadow channel, the current in the east end of Little Hell Gate proper being slack and full of whirlpools, or setting to the east and showing a greater flow through Sunken Meadow Kill than through Little Hell Gate, resulting in a flow entirely around Sunken Meadow, from the East River at the north end back to the East River at the south end. The Sunken Meadow channel could, however, be closed advantageously were the eastern mouth of Little Hell Gate given proper dimensions and direction.

The total discharge of the East River past Mill Rock, as determined from the observations of the Engineer Department in 1910-1911, was, for a mean tide, about 4,656,000,000 cubic feet toward the Sound and about 4,897,000,000 cubic feet toward Sandy Hook, showing a difference of 241,000,000 cubic feet in favor of the southbound tide. These discharges are approximate only and differ somewhat from those previously computed by the United States Coast and Geodetic Survey. The difference may be explained by the fact that the mean stage during the lunations in which the observations were made by the Engineer Department was .3 foot higher than the mean stage at the time the Coast Survey observations were made. As stated, the mean stage differs at different periods of the year. The United States Engineer observations were not continued long enough to determine the true mean stage. A small change in the elevation of the mean stage makes a considerable difference in the total discharge and may make a change in the direction of the resultant discharge. Any error resulting from the stage adopted in this discussion is on the safe side. (See Plates 2B and 3B, pages 257 and 259, respectively, for current directions at the various lunar hours after the moon's transit.)

The observations show some noteworthy features. One is that the water surface at Mill Rock, on the northbound tide, is slightly higher than at 89th Street, New York (Horns Hook), although at those times there is a down slope in the main channel past Mill Rock toward Hallets Point. The water surface in the Harlem River past Wards Island is at those times practically level with that at Mill Rock. Similar conditions obtain during the greater portion

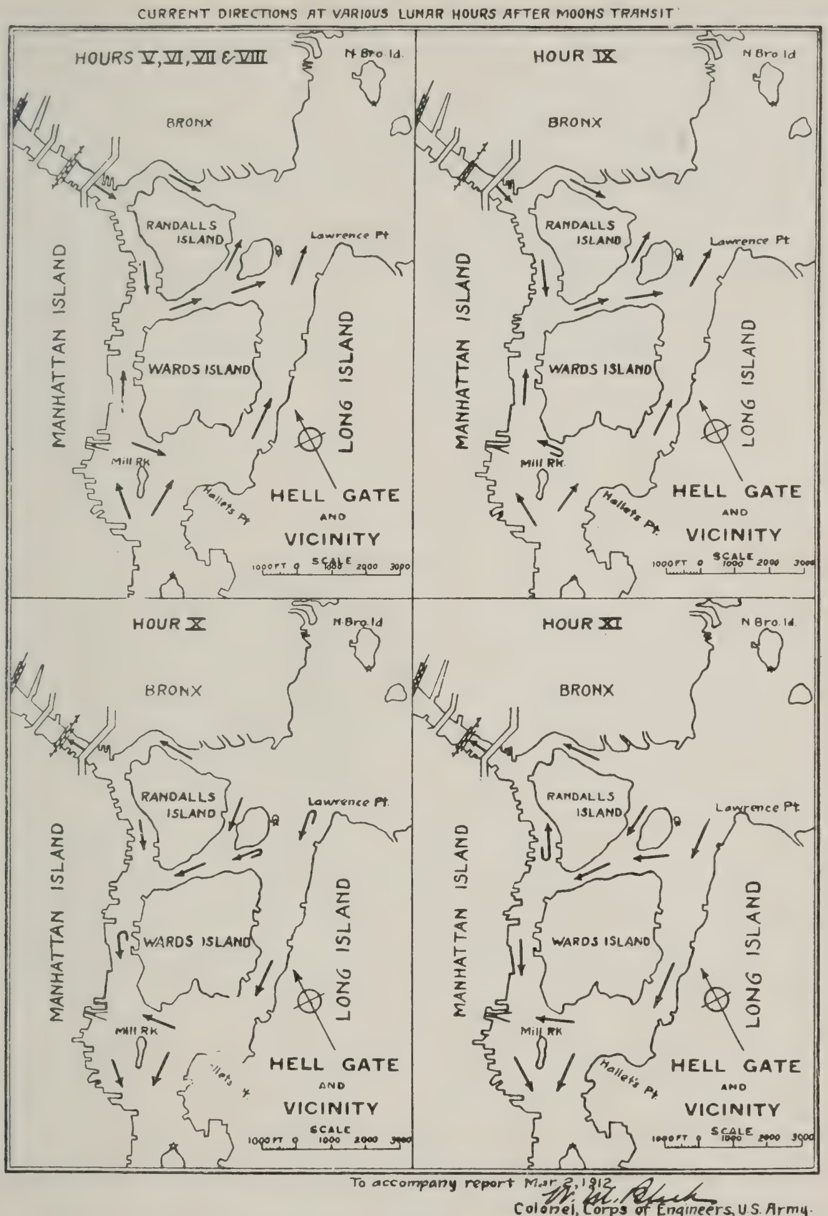


Plate 3B. (See page 258.)

of the southbound flow, the elevation at Mill Rock being higher than in the adjacent main channel. This phenomenon appears to be due to the swing of the current in making the turn through Hell Gate.

Another feature is that during the northbound tide the slope through Hell Gate is for some hours largely concentrated over the submerged reef between Hallets Point and the Hogs Back, the slope from Lawrence Point towards North Brother Island being reversed and showing the water moving uphill. This peculiarity was indicated in all the observations and appears to exist beyond all doubt. (Plate 16B, opposite page 256.) On the southbound tide the slope seems to be concentrated chiefly between Negro Bluff and Hallets Point and the reef does not produce any visible effect.

Another feature is that several places, notably at Lawrence Point, the difference in time between the tide waves of the East River and of Long Island Sound produces a considerable increase in the slopes. Thus, as shown on Plate 2, the northbound tidal currents continue past Lawrence Point until the time marked slack, about an hour before the time of high water at Lawrence Point. The time of high water at Hallets Point marks the time when the Sandy Hook tidal wave crest has reached that point. Through the reach to the south the crest has passed and the water is falling, but the crest of the wave coming from the Sound has been advancing southward, has met the Sandy Hook wave crest north of Hallets Point, overcome it, passed on, and has reached Hallets Point, causing a delay in the fall of the water level there plainly marked on the tidal curve. The effect of this general wave action is to cause a reversal of current to the southward south of Blackwells Island before the incoming Sound wave crest has given head sufficient to overcome the northbound currents at Hallets Point, so that when these latter currents are reversed, steep surface slopes have been formed with high current velocities.

DESCRIPTION OF FIELD AND OFFICE METHODS.

The application of the laws of stream flow to the flow of tidal waters is rendered difficult by the constant change of head produced by the progress of the tidal waves. The variations in height of these waves caused by the varying positions of the moon and sun with respect to the earth cause other confusing variations of head. In New York Harbor further complications are produced by the superposition or interference of the waves entering by Sandy

Hook and Throgs Neck, respectively, and by variations of head caused by changes of the general levels of the Ocean and Sound under wind action.

The records of the automatic tide gauges showed how great were these variations in head. In shallow auxiliary channels like that of Little Hell Gate their effect is magnified in the tidal volumes passed, so that discharge measurements on succeeding days may differ widely.

The force available was too small to permit simultaneous current observations to be made in all of the channels in the vicinity of Hell Gate, for which the regimen had to be determined. It was, therefore, necessary to make the observations when possible, to calculate the discharge for that period, to make careful observations for slopes by a comparison of the various tide gauge records and to determine the relation between discharge and slope for each channel and, finally, reduce the discharge and velocities for each channel to their values for a mean tide with its corresponding slopes.

METHODS.

The field work included observations for tidal slopes and current velocities, employing automatic tide gauges and large Price current meters, supplemented where necessary by tide-staff observations.

The channels of the Harlem and of the East rivers are crowded with shipping, making interruptions frequent even during night work. The great depth and rapid currents of the Hell Gate channel added an element of decided danger, and led to the death by drowning of one of the party. The greater part of the field work was done under the immediate supervision of Mr. A. von Siller, Junior Engineer, and to him and his party great credit is due for the work performed under so many adverse conditions.

Most of the office work and computations and of the last part of the field work was carried on under the immediate supervision of Mr. E. F. Robinson, Junior Engineer. The details of the necessary departures from the methods in general use were worked out successfully by him, and great credit is due him for his work in connection with the solution of the complicated problems involved in this investigation.

TIDAL OBSERVATIONS.

Datum.

All elevations were referred to the Hallets Point plane of mean

low water of 1870, which is the mean of fifty-seven consecutive low waters read upon a staff gauge set in a stilling box.

This plane is 2.437 feet below the plane of mean sea level as established by the United States Coast Survey at Sandy Hook, N. J., and 0.347 foot below the Battery datum of the Dock Department of the City of New York (see Plate 2, opposite page 250).

The base bench-mark of the survey is known as Electric Light Tower, and is a steel bolt let into the concrete base of an electric light tower at Hallets Point, Astoria, L. I. Its elevation is 10.036 feet above the Hallets Point plane of mean low water of 1870.

The tide gauges used were of the Stierle pattern, with one clock for driving and keeping time. They were old, and gave trouble, owing to variations in the force required to operate the recording apparatus and the irregular rates of the clocks. A new gauge was finally procured which had a time clock, which marked each hour by a short horizontal motion of the recording pencil. This apparatus did not always work, and the lines were so drawn as to make an exact determination of the hour point difficult.

An apparatus was devised by Mr. A. von Siller, Junior Engineer, for obtaining accurate time, which consisted of a ship's clock, with electric connections for striking bells. This clock was placed entirely independent of the gauge apparatus, and had its striking hammer in a weak circuit with a pony relay. This relay controlled a stronger electric current, which actuated a magnet on the gauge, causing a second pencil to make a vertical mark upon the record as the clock struck the hours and half hours. The magnets, however, were so constructed as to permit actual contact of the armature, and were liable to stick for a considerable period, all time record being lost in the meantime. Again, the operating current was so strong as to cause sparking at the contact points of the relay, burning them so that they made poor contacts, and the whole apparatus finally refused to work. The observations, however, were completed by this time, and no further opportunity was offered for testing the device.

The records, when brought into the office, were worked up in sets of fifty-seven consecutive tides, this representing one lunation, or complete period of the moon.

This was accomplished in the following manner:

The record was first corrected for time, and each noon and midnight marked with the time of each lunar transit. The zero line

was then corrected according to the notes of the staff readings scattered through the record. The exact time of each high and low water was then determined, as well as the luni-tidal interval, or the elapsed time since the last lunar transit, and the records were then worked up by sub-dividing each tide into appropriate intervals.

The period between successive high and low waters was divided into six equal parts, representing approximately the lunar hours, and the height of the curve from the corrected zero line was measured at each low and high water and intermediate points. These heights were tabulated in columns, one each for low water: first hour after low, second hour . . . high water, etc., for the high and low water luni-tidal intervals, and for the intervals from low water to high, and the reverse. The mean of the fifty-seven items in each column was then taken, and a curve constructed upon these ordinates which represents the mean tide in range but not in slope, this latter being too much affected by the irregularities in the gauge clocks to be reliable.

It was found that the mean curves for one lunation taken at the same point have practically the same range and characteristic shape, whatever the date of observation, but that the elevation of mid-range varies by as much as half a foot for the same locality at different periods. For instance, the mean curve for 112th Street, Harlem River, for March, 1911, had a range of 5.0 feet and a mid-stage height (measured above the low water plane) of 2.38 feet, while for June, 1911, the range was 4.97, and the mid-stage height, 2.96. This mid-stage elevation is hereinafter denominated for convenience "mean height." Again, for *simultaneous* curves taken at different points it was found that the mean height above the low water plane was the same, though this height might vary with the season. In May-June, 1907, a simultaneous lunation was obtained for six points in this district, including the Harlem River and East River, above and below Hell Gate. The mean height of each of these six curves agreed within 0.05 of a foot with the mean of all six. Similarly, the records at a number of points at which it was possible to secure simultaneous curves during the course of the present investigation have shown a like agreement.

Owing to the uncertainties of the automatic tide gauges and to a lack of the proper number, it was impracticable to secure simultaneous lunations at all the required points of observation, and recourse was had to simultaneous tide-staff readings taken at special

points between which it was desired to know the tidal slopes or heads. These readings were taken for various stages of the tide, the dates for the mean tide readings to be used in investigating the effects of a diversion of flow from Hell Gate being determined from the Coast and Geodetic Survey Tide Tables.

The plane of Mean Sea Level, as established by the United States Coast and Geodetic Survey at Sandy Hook, is +2.44 above the Hallets Point Datum of 1870, which was used as the plane of reference for the Engineer Department survey, but very few of the lunations observed had a mean height so low. A mean of all lunations observed during a period of two years gave the required elevation as 2.73 feet, and a mean of all high and low waters for one year at Lawrence Point, representing the East River tides, and for the same year at Willis Avenue Bridge, representing the Harlem River, gave 2.76.

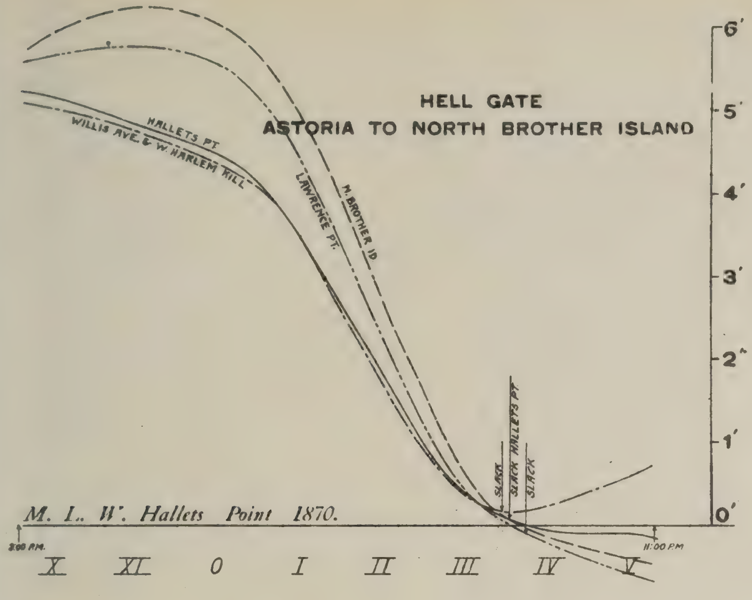
CURRENT OBSERVATIONS.

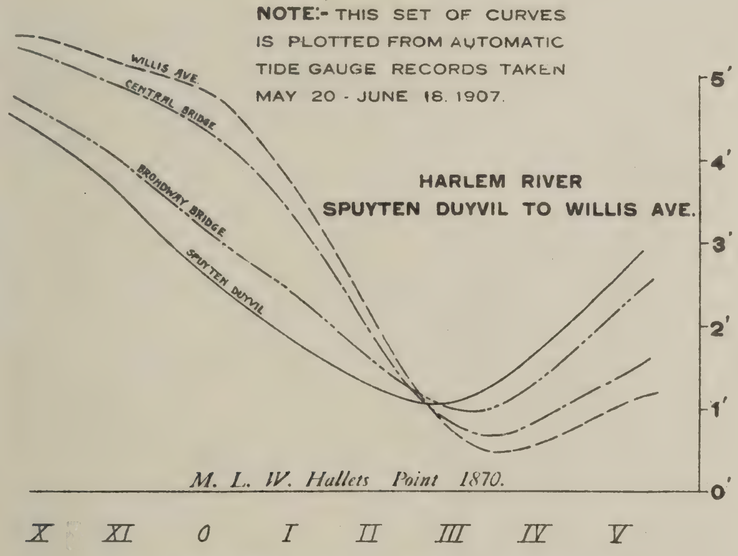
Current observations were taken in Hell Gate, Little Hell Gate, Harlem Kills and the Harlem River (Plate 1, p. 251). The method of procedure in the three latter channels was identical, so that a description of the operations in Little Hell Gate will be typical of all three. In Hell Gate, however, owing to the great depths and rapid currents, a different method had to be devised, the points of variation being in the duration of the observation and the means of locating the distance out and depth of the meters.

In Little Hell Gate, observations were taken continuously for three days and nights. There being no navigation in this channel, a line was stretched between trees on opposite banks, and at three points on this line, 100, 200 and 300 feet, respectively, from the north bank the meters were suspended from blocks by double conductor electric cables, and slid up and down on bronze wire cables attached to the line and to anchors on the bottom. The conductor cables led to the scow *Sweep*, which was anchored about the center of the channel, but far enough from any meter to prevent interference. The channel at the meter section was about 400 feet wide, and comparatively unobstructed for a run of about 200 feet on either side.

Each cable was belayed on the deck of the *Sweep*, which was so graduated that any given length of cable could be paid out or taken in, permitting the placing of the meters at any desired depth. A table was prepared, giving for each half foot of tide the amount of

PLATE 14B





COMPARATIVE TIDAL CURVES
FOR A SOUTH-BOUND TIDE IN HELL GATE AND CONNECTED CHANNELS.

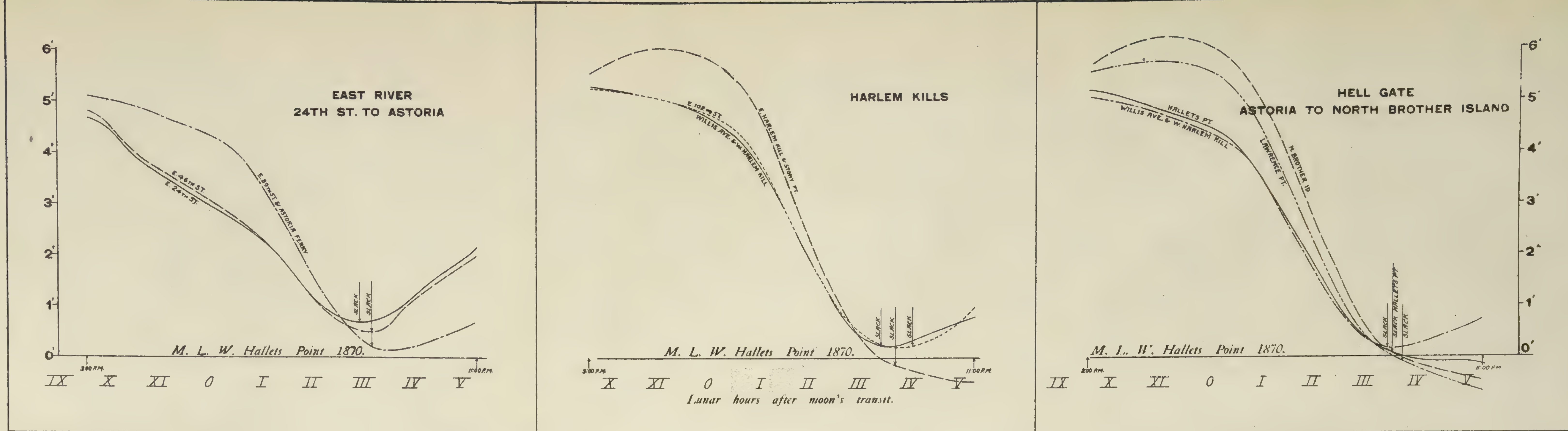
PLOTTED, EXCEPT AS NOTED, FROM TIDE STAFF AND AUTOMATIC GAUGE READINGS TAKEN JULY 20, 1912.

SCALES:- HORIZONTAL 1 IN.= 1 LUNAR HOUR VERTICAL 1 IN.= 1 FT.

TO ACCOMPANY REPORT OF MARCH 2, 1912

W. M. Black

COLONEL CORPS OF ENGINEERS, U. S. ARMY

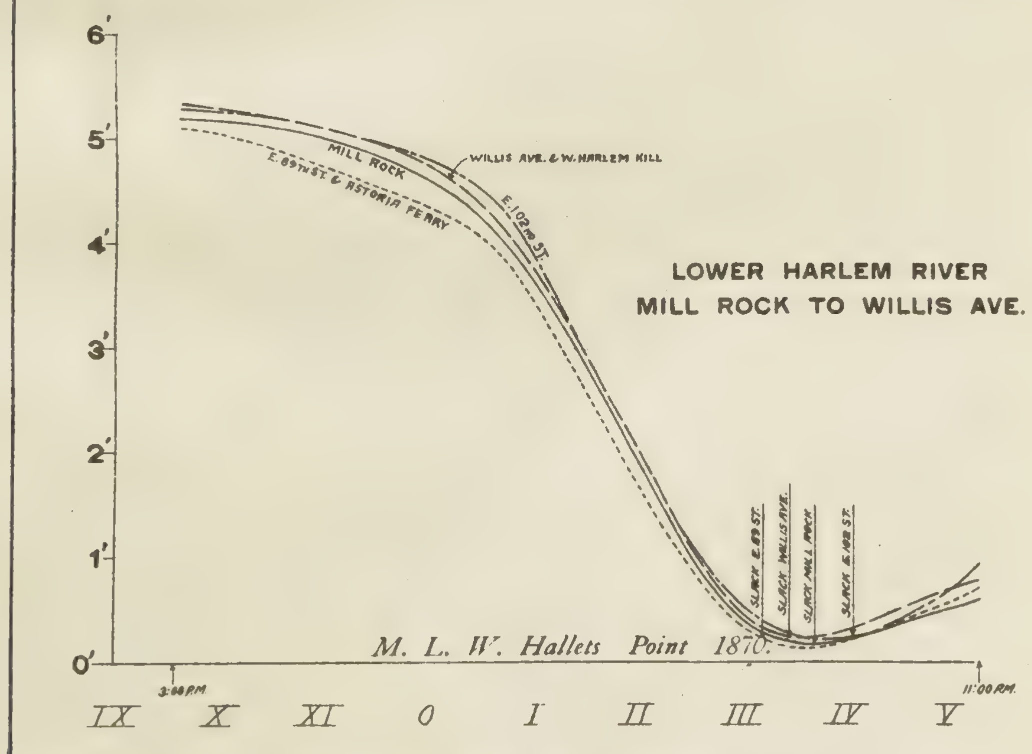


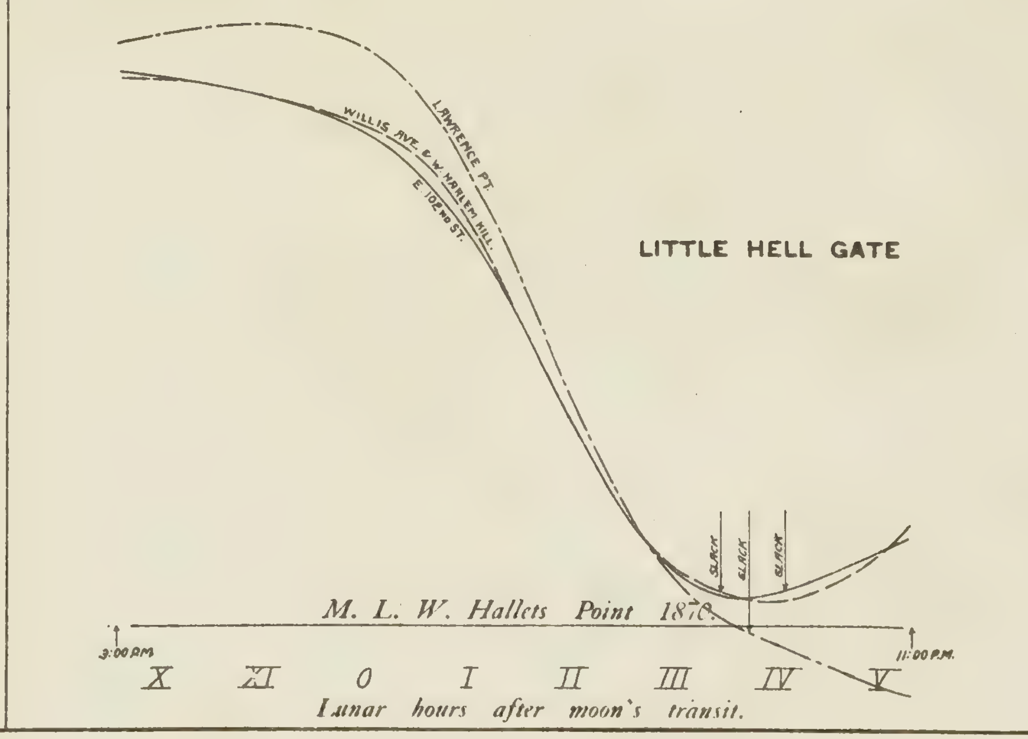
GENERAL NOTES

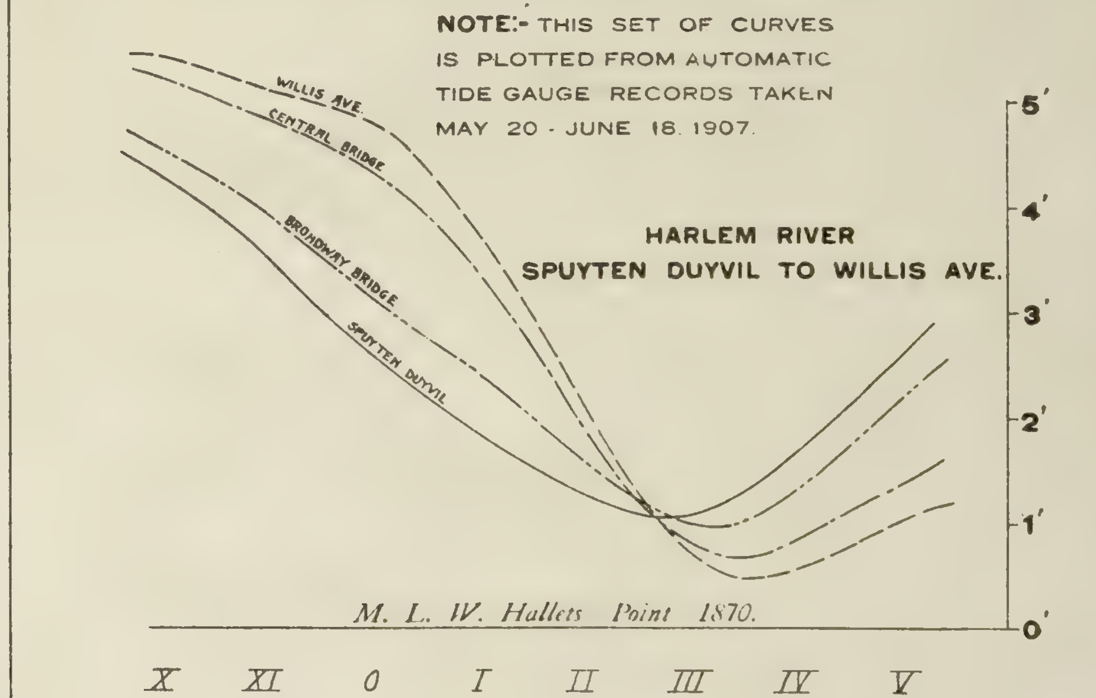
THE TIDE RANGE AT LAWRENCE POINT DURING THESE OBSERVATIONS WAS ABOUT 6.55 FT. THE MEAN TIDE RANGE THERE U. S. C. & G. S. IS 6.60 FT.

THE LOCATION OF TIDE STAFFS AND GAGES IS SHOWN ON PL. 1B.

THE DIFFERENCE OF THE TIDAL HEIGHT AT ANY TIME IS SHOWN BY THE VERTICAL ORDINATES BETWEEN THE CURVES.







COMPARATIVE TIDAL CURVES
FOR A SOUTH-BOUND TIDE IN HELL GATE AND CONNECTED CHANNELS.

PLOTTED, EXCEPT AS NOTED, FROM TIDE STAFF AND AUTOMATIC GAUGE READINGS TAKEN JULY 20, 1912.

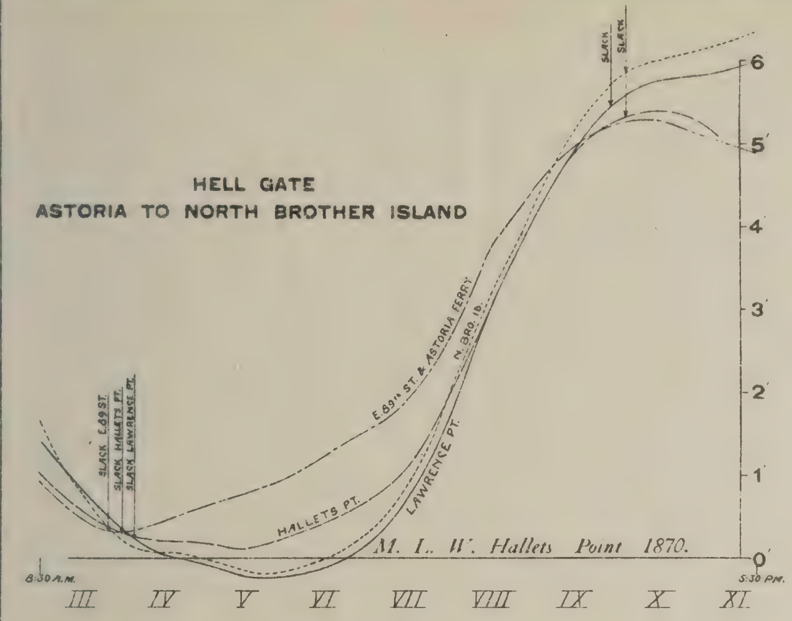
SCALES:- HORIZONTAL 1 IN. = 1 LUNAR HOUR VERTICAL 1 IN. = 1 FT.

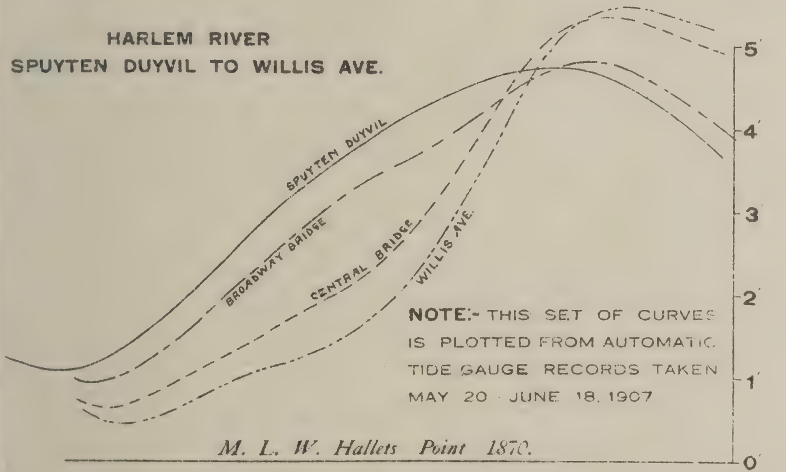
TO ACCOMPANY REPORT OF MARCH 2, 1912

W. M. Black

COLONEL CORPS OF ENGINEERS, U. S. ARMY

PLATE 15B





COMPARATIVE TIDAL CURVES

FOR A NORTH-BOUND TIDE IN HELL GATE AND CONNECTED CHANNELS.

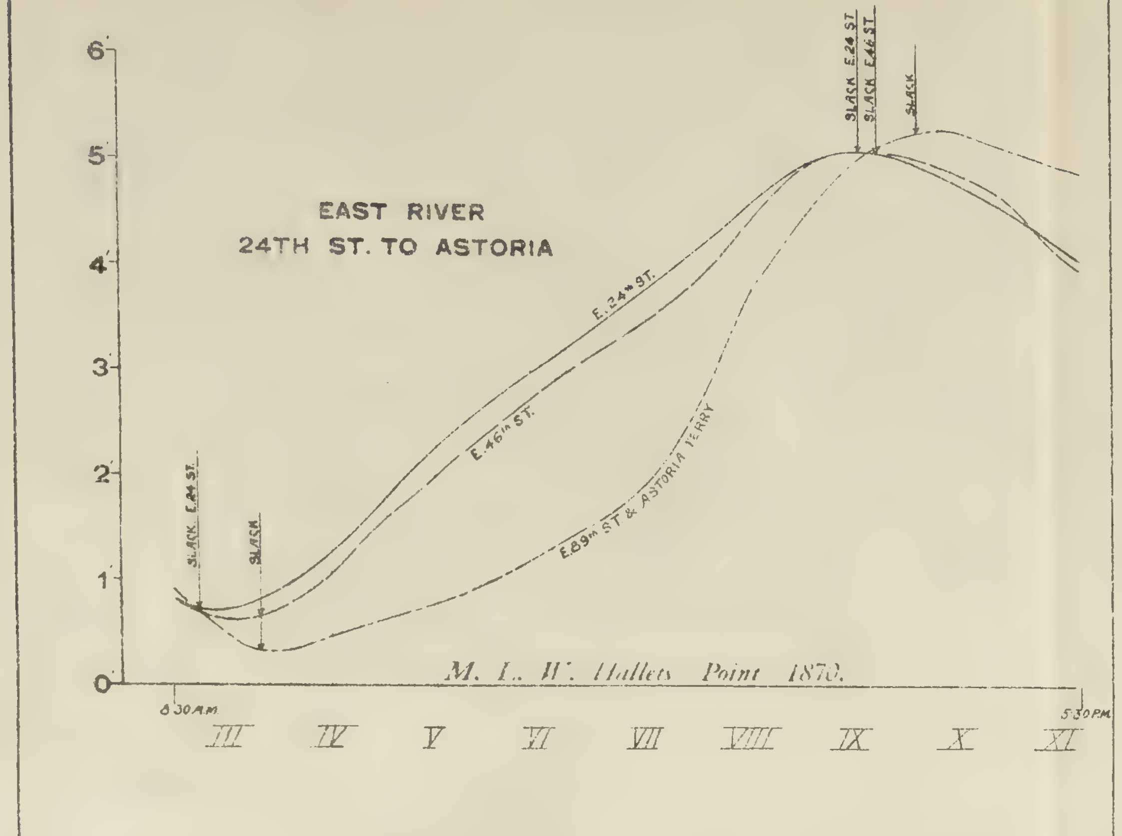
PLOTTED, EXCEPT AS NOTED, FROM TIDE STAFF AND AUTOMATIC GAUGE READINGS TAKEN JULY 21, 1912.

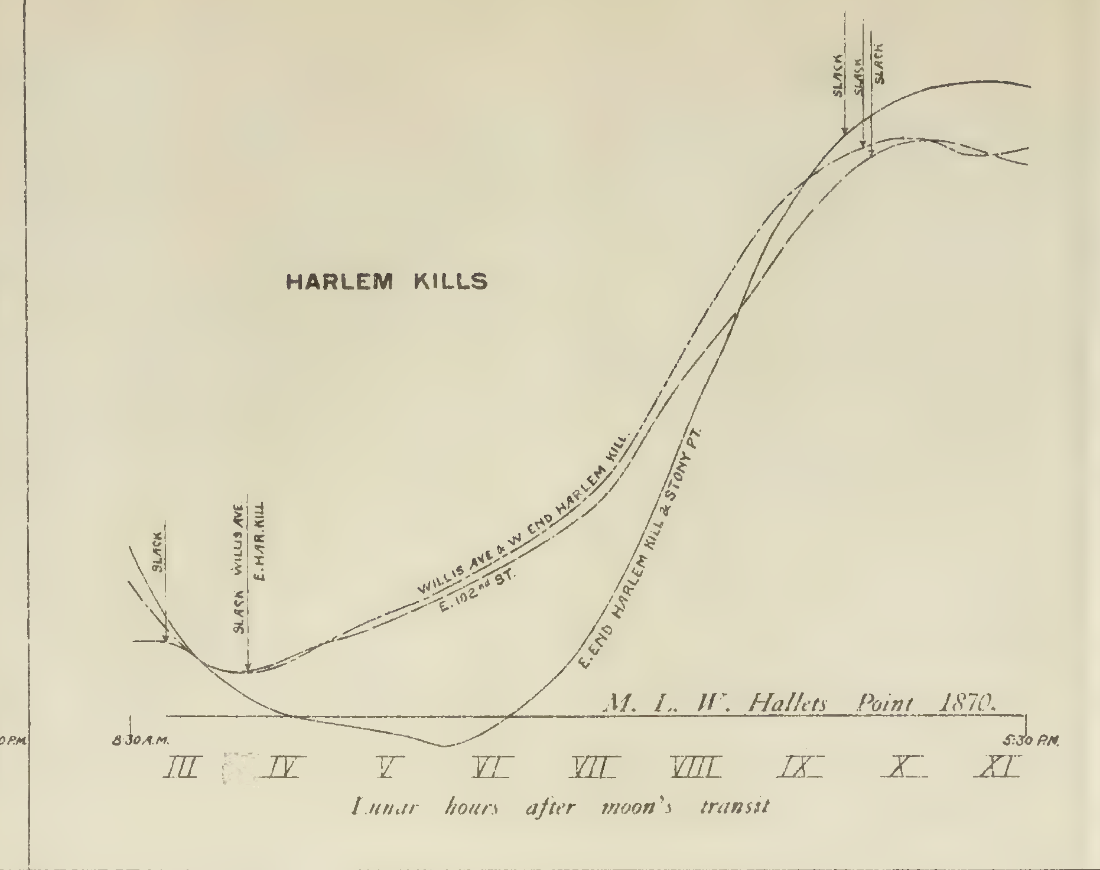
SCALES—HORIZONTAL, 1 IN.=1 LUNAR HOUR; VERTICAL, 1 IN.=1 FT.

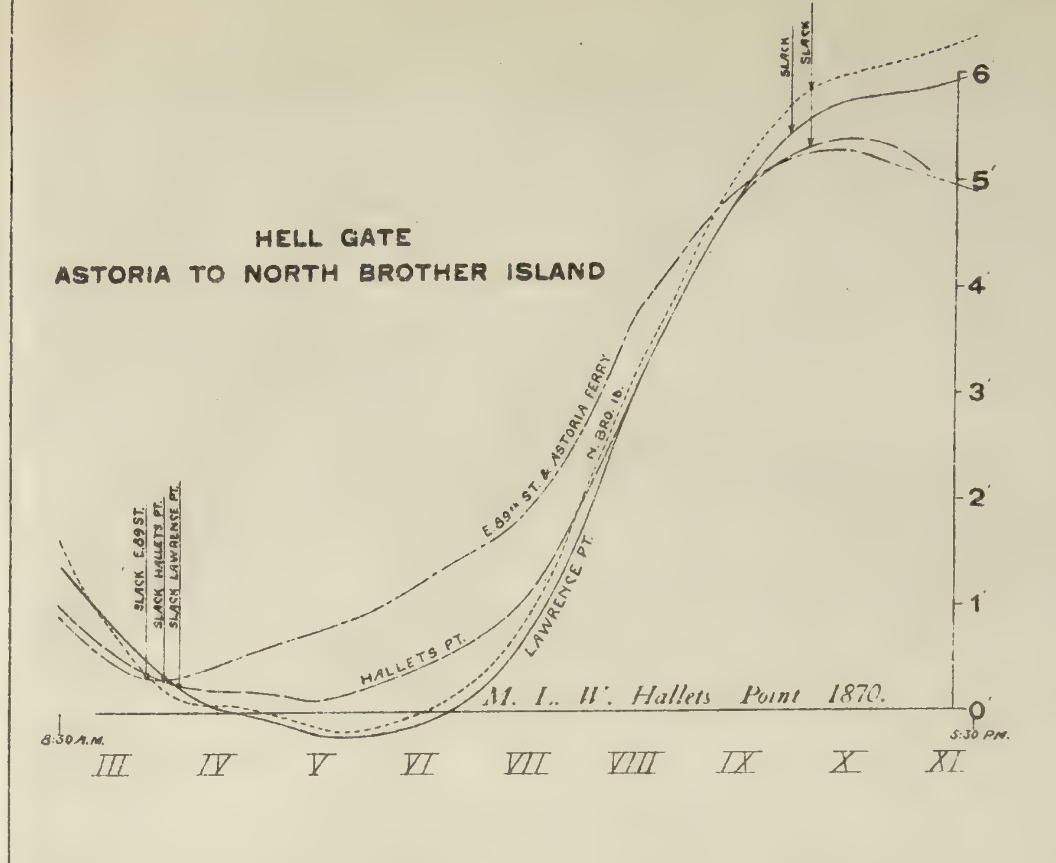
TO ACCOMPANY REPORT OF MARCH 2, 1912

W. M. Black

COLONEL CORPS OF ENGINEERS, U. S. ARMY





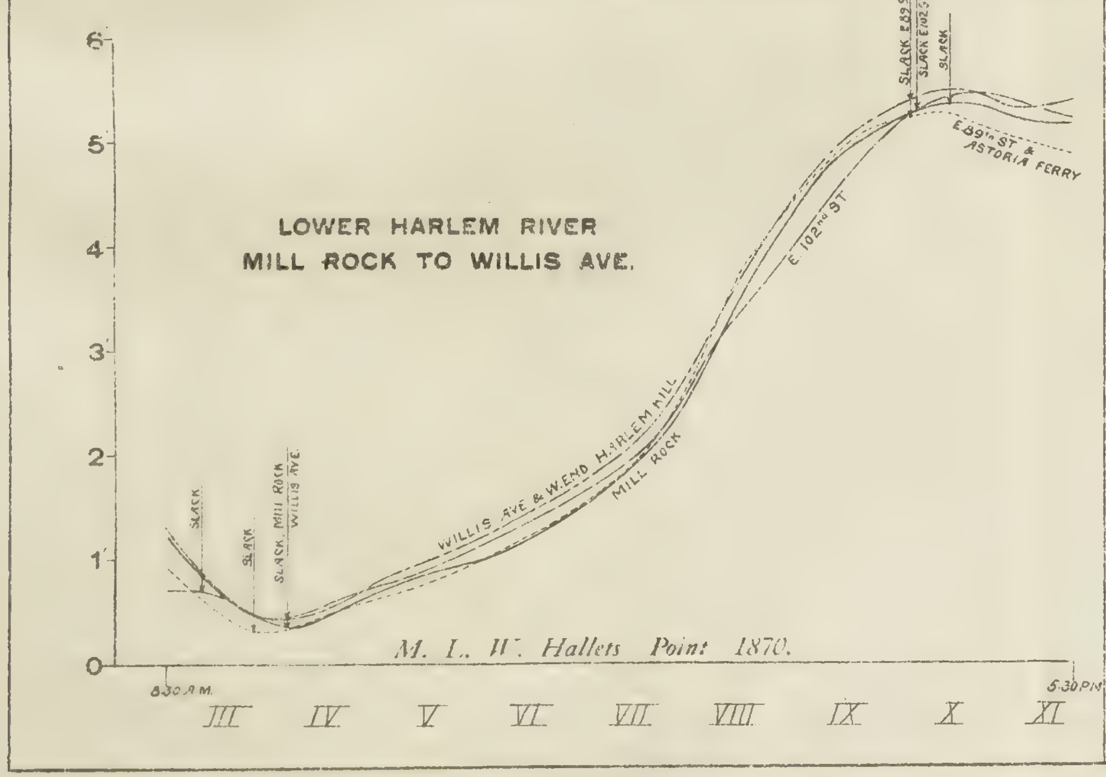


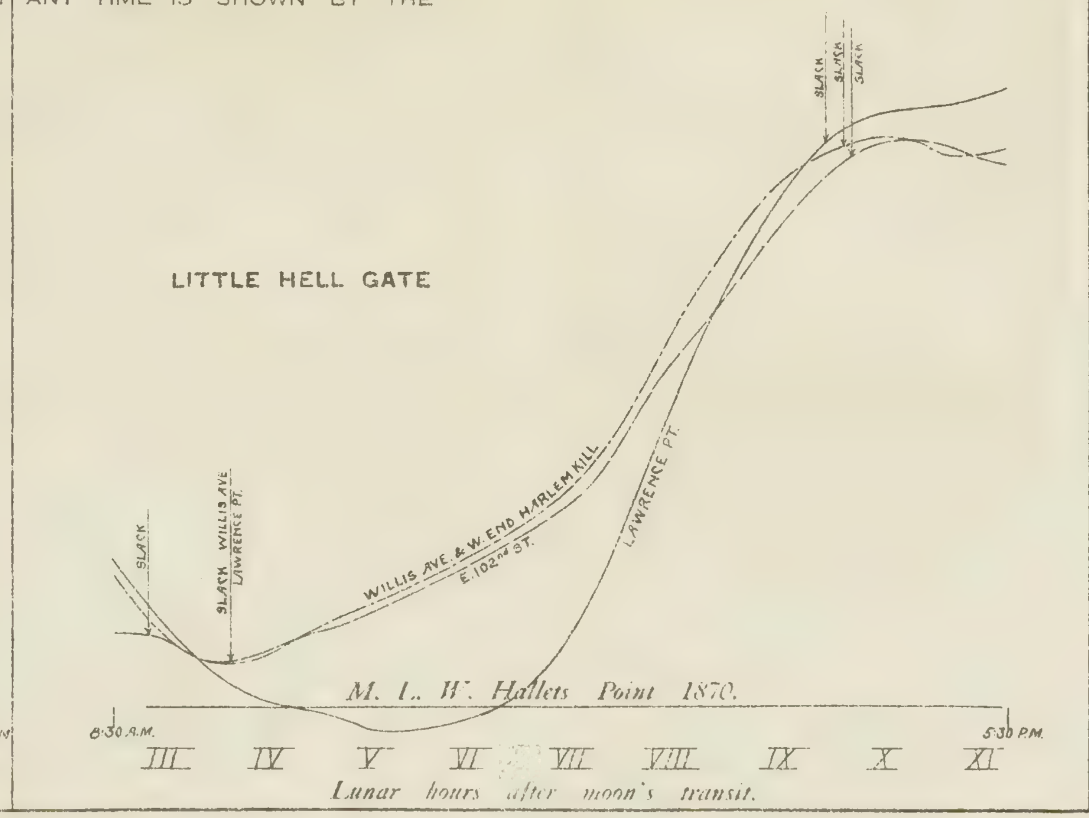
GENERAL NOTES

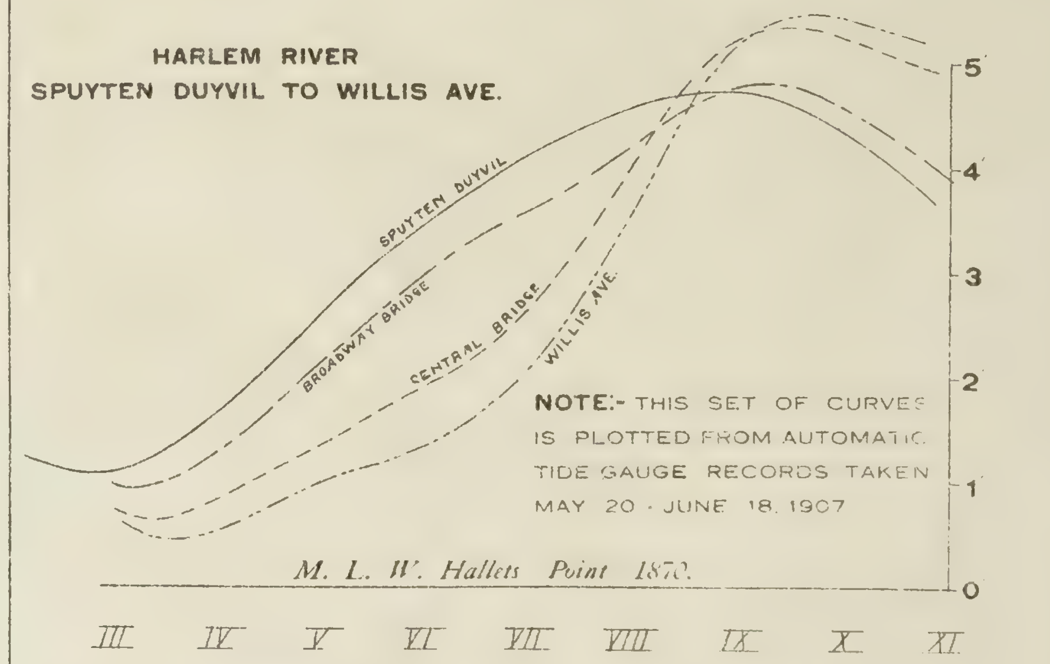
THE TIDE RANGE AT LAWRENCE POINT DURING THESE OBSERVATIONS WAS ABOUT 6.55 FT. THE MEAN TIDE RANGE THERE (U. S. C. & G. S.) IS 6.60 FT.

THE LOCATION OF TIDE-STAFFS AND GAUGES IS SHOWN ON PL. 1B.

THE DIFFERENCE OF THE TIDAL HEIGHT AT ANY TIME IS SHOWN BY THE VERTICAL ORDINATES BETWEEN THE CURVES.







COMPARATIVE TIDAL CURVES
FOR A NORTH-BOUND TIDE IN HELL GATE AND CONNECTED CHANNELS.

PLOTTED, EXCEPT AS NOTED, FROM TIDE STAFF AND AUTOMATIC GAUGE READINGS TAKEN JULY 21 1912.

SCALES:—HORIZONTAL, 1 IN.= 1 LUNAR HOUR; VERTICAL, 1 IN.= 1 FT.

TO ACCOMPANY REPORT OF MARCH 2, 1912

W. M. Black

COLONEL CORPS OF ENGINEERS, U. S. ARMY

each cable to pay out in order to place the meters at 0.2, 0.6, or 0.8 of the depth. The cable connections led into the cabin of the *Sweep*, terminating in telephone head receivers, in which the grating sound of the contact was reproduced at each revolution of the meter wheel.

Observations were made at intervals of ten minutes, the observer taking each meter in turn and timing ten revolutions (twenty near slack water) with a stop-watch. The deck hands outside regulated the depth of the meters as directed by the observer, using the table mentioned above. Tide was observed from a staff gauge on shore, read with a field glass, and (at night) an acetylene searchlight. Each observation, then, consisted of nine readings, three depths for each meter, the changes of depth being effected during the reading of some other meter. Between observations the observer and recorder, with a slide rule and rating curve, reduced each reading to revolutions per second, obtained the mean in the vertical by the formula

$$M = \frac{\text{No. Rev. for } 0.2 + 2\ (\text{for } 0.6) + \text{for } 0.8 \text{ depth}}{4}$$

and determined the velocity in feet per second.

In case of a breakdown where no record from some particular meter could be obtained, a deckhand went out in a small boat, dislodged the débris, if any, or brought the meter back for examination and testing of its connections. The methods were identical in Harlem Kills and the Harlem River, except that in the latter stream the supporting cable led only from the scow *Sweep* to one shore, leaving one-third of the channel open for navigation. Current directions were observed by means of independent floats.

REDUCTION.

To reduce the observations, the first step was the preparation of what has, for the lack of a better name, been designated the local velocity curve, which is for each meter the graphical record of the rise and fall of the velocity at the meter point. This curve passes through zero at slackwater, becomes alternatively positive and negative as the direction of the current changes, and partakes of the general shape of the sine curve. Its use has the great advantage that any break in the record of not more than about two hours'

duration may be bridged over and the record made continuous. (Plate 5B.*)

The hourly horizontal-velocity curve for the section was constructed by plotting at the proper position for each meter, the velocity at the middle of the hour as shown by the local velocity curves for that position. Through the three equally spaced points thus determined, the curve is drawn for the entire width of the channel. To obtain the discharge rate, equally spaced ordinates were drawn to the horizontal velocity curve and their lengths were multiplied by the depth ordinates at the same points, correcting these latter for the mid-hour tide. The sum of these products, multiplied by the interval between ordinates, gave the area of a curve whose ordinates are the product of depth by velocity, or the rate of discharge.

The discharge curve is drawn in a manner similar to the local velocity curves. The abscissæ are hours. At each mid-hour point is erected an ordinate representing the rate of discharge as determined for that moment. The extremities of these ordinates are connected by a curve. If the local velocity curves show marked irregularities between these points, determinations may be made at closer intervals. (Plate 6B, see page 253.)

Upon the same time ordinates, but lower in the sheet, the tidal curve for the period is drawn, together with the area curve, the latter denoting at any time the area of cross section due to the tidal height at that moment. Dividing any discharge ordinate by the simultaneous area ordinate gives the ordinate to a fourth curve, that of mean velocity, which is plotted upon the same axis as the discharge curve. The area included between the discharge curve, the horizontal axis and any two time ordinates, represents the volume of discharge for the included period.

Finally, there is drawn upon a third axis and upon the same time ordinates, the tidal curves for the extremities of the channel, as registered by the automatic tide gauges during the current observations. Vertical ordinates subtended between these two curves represent the tidal head, causing the rate of discharge represented by that portion of the *same* time ordinate subtended between the discharge curve and its horizontal axis.

In attempting, however, to relate the discharge to this head, it is

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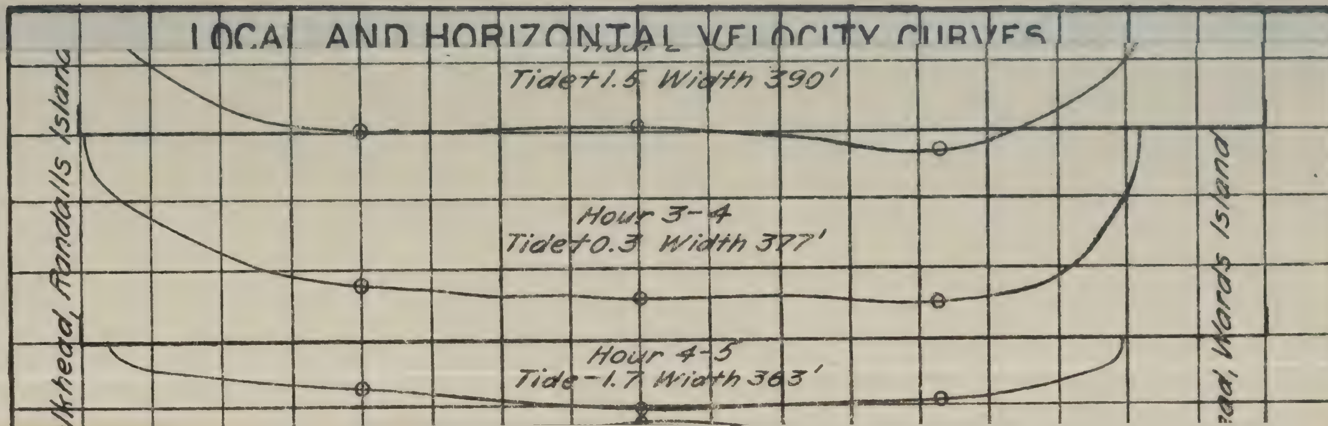
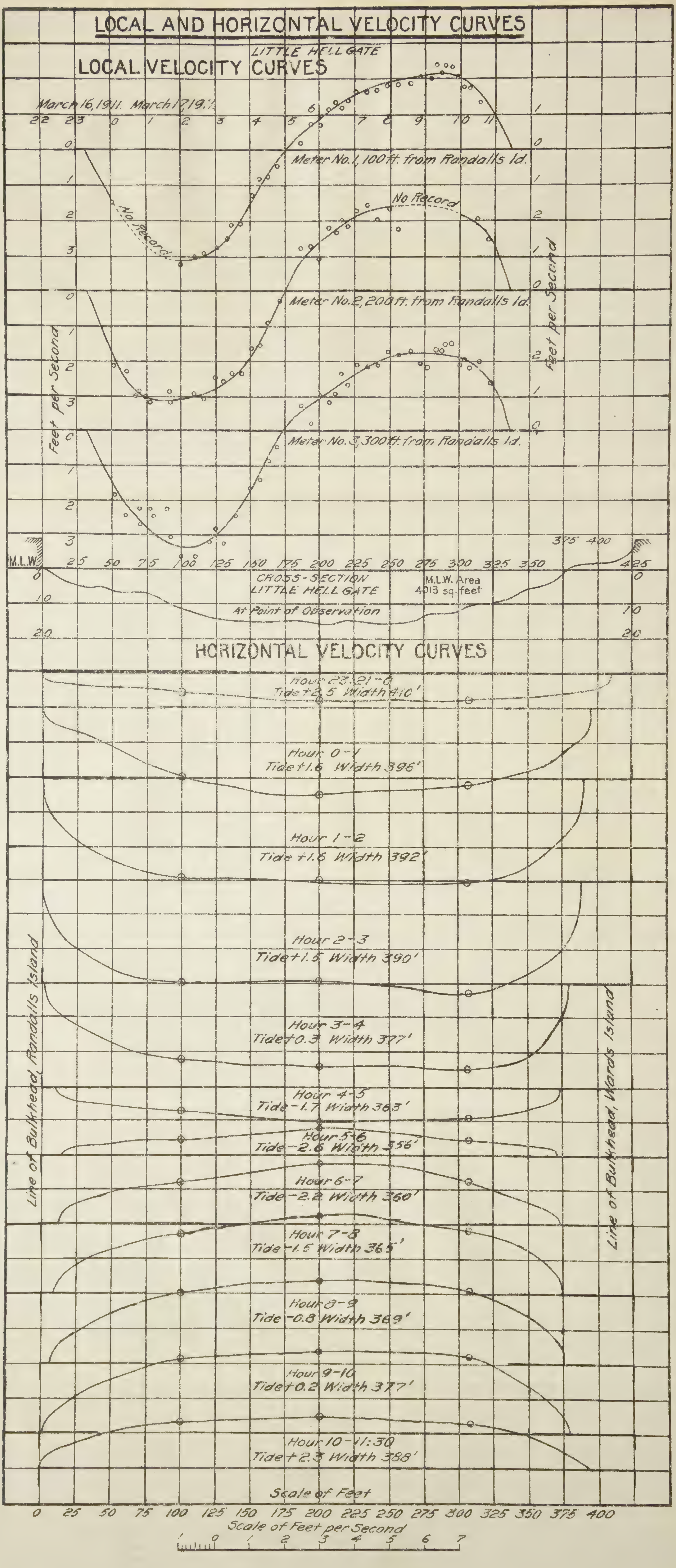
*The second curve under horizontal velocity curves should read tide +2.0 instead of +1.6; the 12th or last curve should extend to 413 feet and the tide should be changed to 2.8 instead of 2.3 and the width to 413 instead of 388.


LOCAL AND HORIZONTAL VELOCITY CURVES
Tide+1.5 Width 390'
Hour 3-4
Tide+0.3 Width 377'
Hour 4-5
Tide-1.7 Width 363'
lkhead, Randalls Island
ead, Wards Island



LOCAL AND HORIZONTAL VELOCITY CURVES
LITTLE HELL GATE
LOCAL VELOCITY CURVES
March 16, 1911. March 17, 19.11.
Meter No. 1, 100 ft. from Randalls Id.
No Record
Meter No. 2, 200 ft. from Randalls Id.
Meter No. 3, 300 ft. from Randalls Id.
Feet per Second
M.L.W.
CROSS-SECTION
LITTLE HELL GATE
M.L.W. Area
4013 sq. feet
At Point of Observation
HORIZONTAL VELOCITY CURVES
Hour 23:21-0
Tide +2.5 Width 410'
Hour 0-1
Tide +1.6 Width 396'
Hour 1-2
Tide +1.6 Width 392'
Hour 2-3
Tide +1.5 Width 390'
Hour 3-4
Tide +0.3 Width 377'
Hour 4-5
Tide -1.7 Width 363'
Hour 5-6
Tide -2.6 Width 356'
Hour 6-7
Tide -2.2 Width 360'
Hour 7-8
Tide -1.5 Width 365'
Hour 8-9
Tide -0.8 Width 369'
Hour 9-10
Tide +0.2 Width 377'
Hour 10-11:30
Tide +2.3 Width 388'
Line of Bulkhead, Randalls Island
Line of Bulkhead, Wards Island
Scale of Feet
Scale of Feet per Second

noted that at the moment of zero slope, that is, where the tidal curves cross, the current is still running, and slack water does not occur until some time later. This phenomenon is due to the momentum of the moving water, which maintains the velocity even after the slope has reversed in direction. It does not appear at each tide to the same extent, but seems to vary with the range of the tide, the influence of winds, etc. The *effective* head, then, becomes zero at slack water, and for a period before or after slack the effective, instead of the actual head, must be considered.

In Plate 6B (page 253), the dotted lines either side of slack water show the shape that the 112th St. curve would assume were the head at all times equal to the theoretical head required to cause the observed rate of discharge. The effective head is measured from the base curve, in this case Lawrence Point, to the 112th Street curve, except near slack water, where it is measured to the dotted correction curve. The effective head, then, is seen to be greater than the actual before slack, owing to the momentum of the moving water, and, by reason of the inertia of the water on the change of tide, to lag behind the actual head for a considerable period after slack until the velocity has picked up.

In order to relate the discharge to the slope which produces it, recourse was had to a graphical diagram, compiled from the observed data in the following manner:

The tidal head corrected for momentum, as explained above, is the varying factor in the discharge formula for a constant channel section and may be used in a graphical diagram in place of the slope which results from the head. A table is prepared for each channel, giving for the entire period of the observation at intervals of about fifteen minutes the height of tide at Lawrence Point (the base curve from which the variations in channel cross section can be found), the head, and the observed rate of discharge. The plotting is made upon logarithmic cross section paper, which has the great advantage for work of this character, that the "curves" become straight lines and are parallel, so that the determination of two points (for direction) on one line and one each on the remaining lines, will complete the diagram. The actual number of points plotted are, of course, much in excess of this minimum requirement.

Observed rates of discharge for that channel are abscissæ and heads are ordinates. A point is marked at the intersection of an abscissa showing a measured discharge at a given time with the ordinate showing the head at that time, and at this point is written the

corresponding height of tide at Lawrence Point. For example: At 0 hours 45 minutes on March 17, 1911, the rate of discharge was 10,800 second-feet, the effective head was 0.51 foot, and the height of tide at Lawrence Point was 2.00 feet. This latter figure is plotted at the intersection of the ordinate and abscissa through discharge 10,800 and head 0.51, respectively. Upon the completion of the plotting, lines are drawn through the plotted figures connecting those that show the same tidal height at Lawrence Point. (See Plates 7B and 8B.)

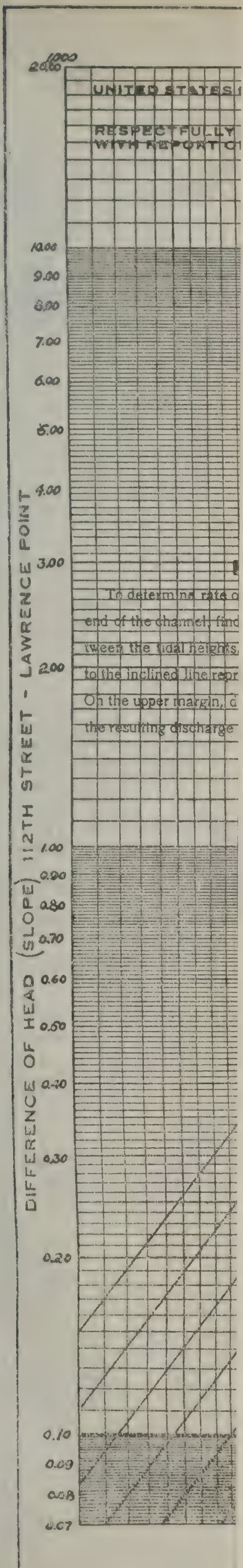
To find the rate of discharge through Little Hell Gate, therefore, it is necessary to know only the tidal heights at Lawrence Point and 112th Street. On the discharge diagram for Little Hell Gate the abscissa corresponding to the difference of height (which, of course, is subject to correction near slack water) is followed horizontally to the plotted line representing the given height of tide at Lawrence Point. The ordinate through this point is followed vertically to the upper margin, where the discharge rate is read off.

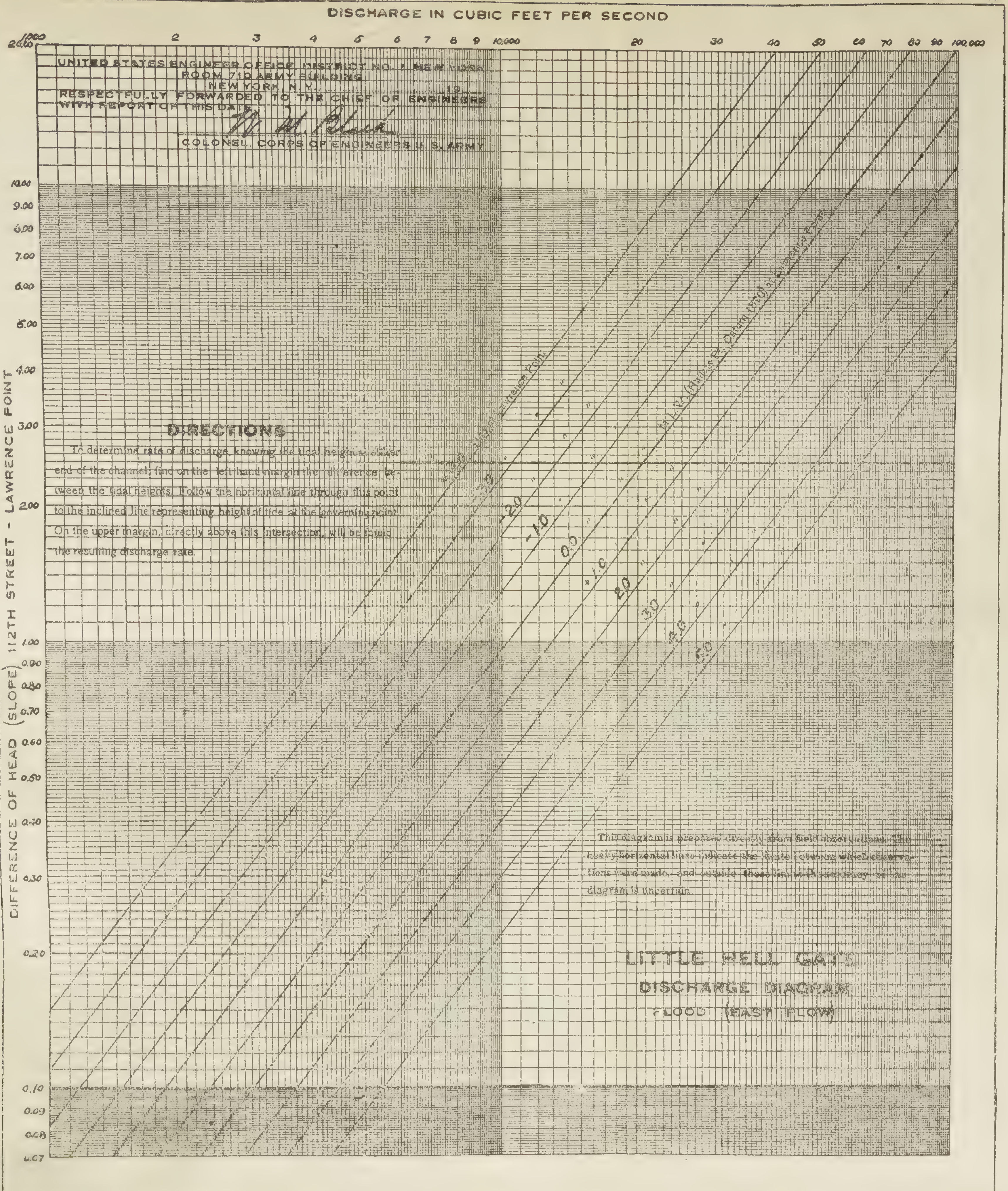
Similarly, diagrams are constructed for Harlem Kills, the Harlem River, and, as will be shown later, for Hell Gate. It is found that separate diagrams are required for the north and southbound tides in each channel. This is due principally to the fact that the same base curve is used for computing both flood and ebb discharge, so that, for the same difference of head and the same height at Lawrence Point, the stage or mean height is greater for the northbound tide. But even when the height at Lawrence Point on the northbound tide is lowered so that the same slope and stage prevails as for the southbound, the resulting discharges are still widely divergent, showing that other factors, such as the different channels of approach, etc., enter largely into the problem. The diagrams are correct, since the points are found from observed data and not from computations, so that in the diagram discharges all of the various factors have entered, and these factors do not change the relative volumes of discharge in the same direction but at different stages and slopes.

HELL GATE.

On account of the great depths and strong currents encountered, together with the constant traffic, consisting largely of car floats and unwieldly tows, often under very poor control, an entirely different method was adopted for the field work in Hell Gate.

The first necessity was a carefully determined cross section at the gorge between Negro Point Bluff and Wards Island and


UNITED STATES
RESPECTFULLY
WITH REPORT
To determine rate o
end of the channel, find
tween the tidal heights,
to the inclined line repr
On the upper margin, d
the resulting discharge
DIFFERENCE OF HEAD (SLOPE) 112TH STREET - LAWRENCE POINT
20.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.09
0.08
0.07


DISCHARGE IN CUBIC FEET PER SECOND
1,000
2
3
4
5
6
7
8
9
10,000
20
30
40
50
60
70
80
90
100,000
UNITED STATES ENGINEER OFFICE DISTRICT NO. 1 NEW YORK
ROOM 710 ARMY BUILDING
NEW YORK, N.Y. 19
RESPECTFULLY FORWARDED TO THE CHIEF OF ENGINEERS
WITH REPORT OF THIS DATE
COLONEL, CORPS OF ENGINEERS U.S. ARMY
DIFFERENCE OF HEAD (SLOPE) 112TH STREET - LAWRENCE POINT
20.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.09
0.08
0.07
DIRECTIONS
To determine rate of discharge, knowing the tidal heights at either end of the channel, find on the left hand margin the difference between the tidal heights. Follow the horizontal line through this point to the inclined line representing height of tide at the governing point. On the upper margin, directly above this intersection, will be found the resulting discharge rate.
-3.0
-2.0
-1.0
0.0
+1.0
2.0
3.0
4.0
5.0
This diagram is prepared directly from field observations. The heavy horizontal lines indicate the limits between which observations were made, and outside these limits the accuracy of the diagram is uncertain.
LITTLE HELL GATE
DISCHARGE DIAGRAM
FLOOD (EAST FLOW)

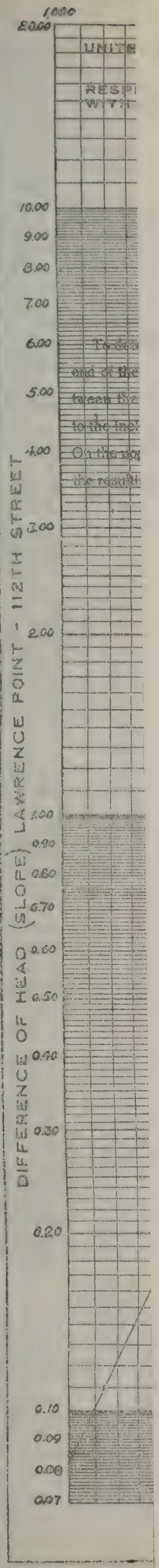
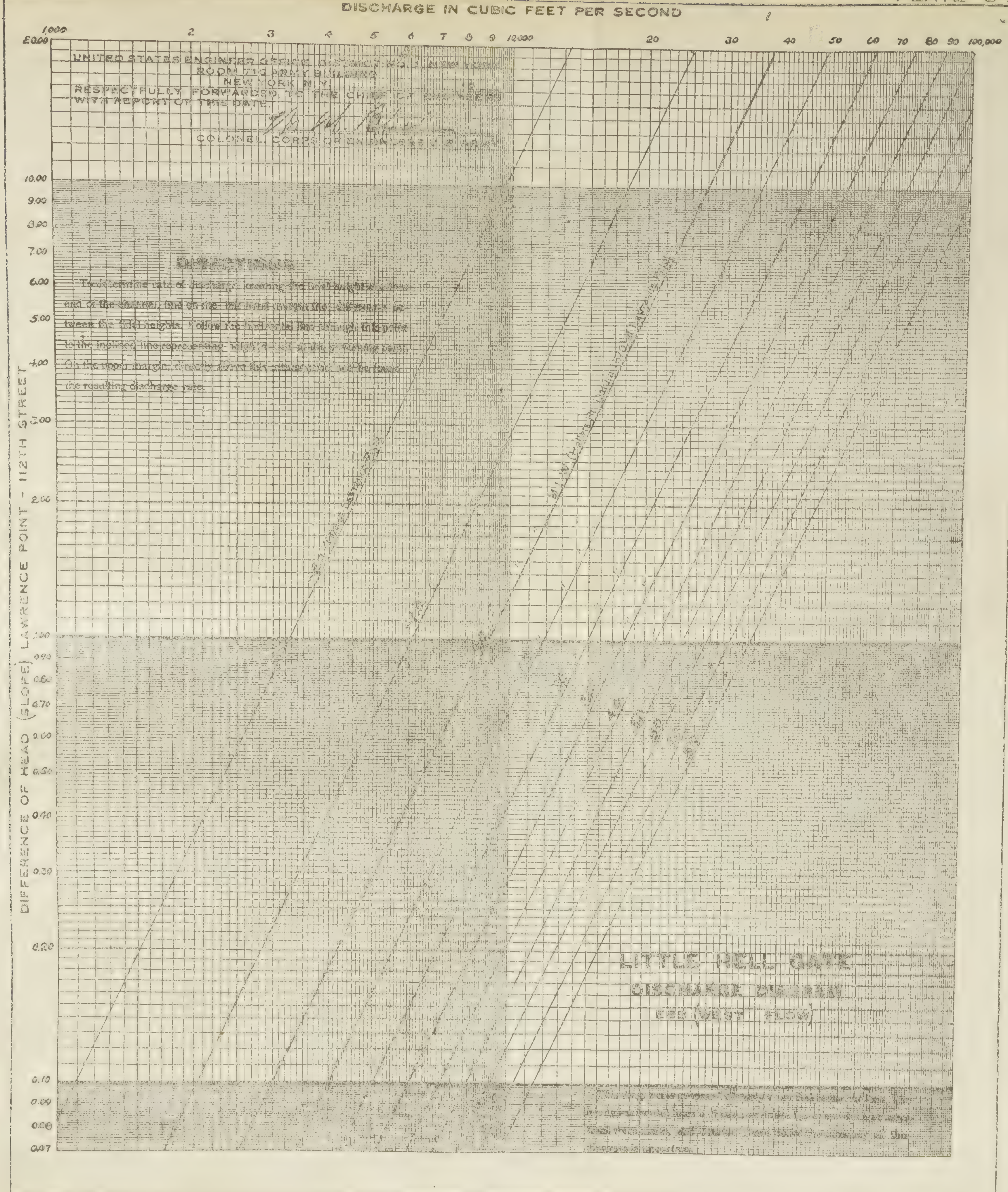

DIFFERENCE OF HEAD (SLOPE) LAWRENCE POINT - 112TH STREET
20.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.09
0.08
0.07

PLATE 8-B


DISCHARGE IN CUBIC FEET PER SECOND
1,000
2
3
4
5
6
7
8
9
10,000
20
30
40
50
60
70
80
90
100,000
UNITED STATES ENGINEER OFFICE
NEW YORK N.Y.
RESPECTFULLY FORWARDED TO THE CHIEF OF ENGINEERS
WITH REPORT OF THIS DATE.
COLONEL, CORPS OF ENGINEERS
20.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.09
0.08
0.07
DIFFERENCE OF HEAD (SLOPE) LAWRENCE POINT - 112TH STREET
the resulting discharge rate.
LITTLE HELL GATE
FLOW)

Sealy Rock on the Long Island shore. The sounding obviously had to be done at slack water, the duration of which in Hell Gate is extremely short, requiring a heavy lead for sounding and considerable speed in its manipulation.

The apparatus shown (Fig. 2b, p. 271) was devised to meet these conditions. The sounding line, of 1-16 inch steel "aviator cord," supported the 20-pound lead, and was carried over three wheels, as shown. The center wheel was exactly 1 foot in circumference and was direct-connected to a cyclometer, which registered tenths of a revolution. As one revolution of the wheel let out 1 foot of line,

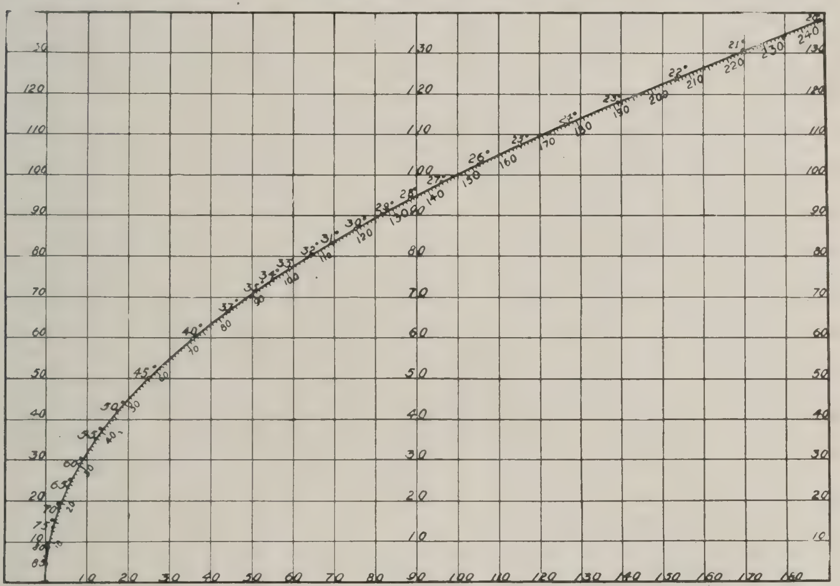


Plate 13B (p. 272). Depth diagram. To determine true depth of current meters or sounding lead, when length and slope of suspending cable are known. Find, on the parabolic curve, the given angle, which represents the slope with the horizontal of a tangent at this point. Determine the ratio between the vertical ordinate to the curve and the length of curve from the vertex to this same point. Multiply the observed length of lead line by this ratio; the result will be the required depth. In a similar manner, using the ratio between horizontal distance and length of curve, the departures of the meters or sounding lead from a vertical may be ascertained.

soundings could be taken to one-tenth of a foot. The cord led to a crab-winch in the boat, geared so as to take in or pay out the line very rapidly.

In operation the boat was rowed across on range, at slack water, and the lead lowered by the winch until bottom was touched, when

the cyclometer was read and a flag signal given to a transitman on shore to cut in on the range and locate the sounding. One range across, with soundings at intervals of about 30 feet, occupied ten or twelve minutes and was all that could be accomplished at one slack period. The proposed meter section was sounded several times, and when plotted the soundings taken in one direction gave the section shown by dotted line (Fig. 3b, p. 273), while those taken in the opposite direction gave a section as shown by the broken line. This was due to the "drag" of the long lead-line while being drawn across behind the boat; and the mean of the two, shown in a heavy line in the figure, was adopted as the true section. Sections were sounded 250 feet above and below the meter section, and found to differ very little from the latter or from each other, so that a reasonably straight run was assured on either side.

The first attempt at measuring current velocities, June 23, 1911, resulted disastrously. A power launch, fitted with tackle for handling meters and the necessary electric recording apparatus, was taken into Hell Gate and an effort made to anchor in the current. The moorings would not hold, and in a struggle to refasten the line one man was dragged overboard and drowned, while some of the apparatus was smashed, necessitating another period of preparation.

The second attempt also failed, owing to the parting of some of the electric conductors leading to the meters, but the *modus operandi* was the same as later succeeded, and which will be described.

It was recognized, as a result of the accident, that any stationary point of observation was impracticable, both on account of the difficulty of holding any craft in the current and of the time that must elapse between observations, in casting loose, moving, and making fast. Interference by traffic, much of which, when caught in the grip of the current is virtually beyond control, must also be considered.

The plan finally adopted contemplated the use of a steam tug, running free, which could be brought slowly up to the range and her engines throttled until she hung poised in the current during the period of the observation, about one minute for each of four depths at one position. No difficulty was experienced in thus holding the tug upon the range and the distance out was determined by a sextant angle read by an observer in the pilot house.

The three meters, heavily weighted, were suspended from an outrigger over the bow by an especially manufactured cable, made up

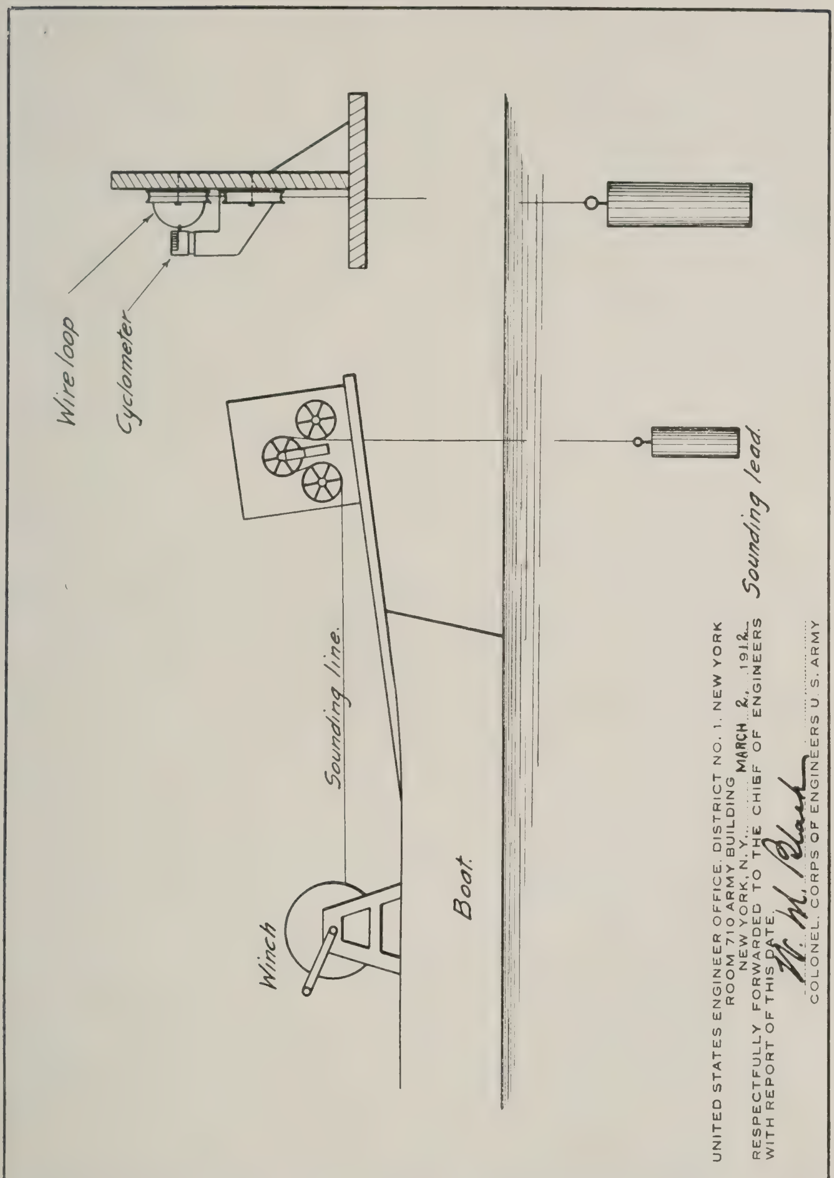


Fig. 2b. Sounding apparatus for taking soundings in Hell Gate. (See p. 269.)

of a center tension member (steel) and six surrounding single-strand electric conductors, insulated from the central core and from one another. Each meter had, therefore, a complete circuit of its own, and the distance between meters (about 1½ feet) was so small that the mean of the three readings could be taken for a position, and if one, or even two, meters stopped recording, a record was still obtainable. The cable led inboard to a winch, the drum of which had been built up by rolling upon it a piece of thick canvas until its circumference was just 1 foot, tested by winding up a 100-foot steel tape in 100 revolutions. The length of cable out was read from a cyclometer, placed on the winch, and reading feet and tenths. The actual depth, however, on account of the swift current, differed greatly from the length of line paid out. Where any correction at all is made for this it is usual to simply measure the angle of departure of the suspending line from the vertical, and to multiply the length of line by the cosine of this angle to determine the true depth.

In Hell Gate, however, this departure was so large, amounting in some cases to 60 degrees, the depth so great (120 feet in the center) and the previous assumption, that the line is straight from the point of suspension, is so manifestly incorrect that a more accurate method of determining the depth was sought.

The line will evidently assume the shape of a curve, due to the tension of the weight acting downward, of the opposite reaction R at the point of support, and of the horizontal force of the current. Were this latter force uniform, the curve would evidently be a parabola, and it may be assumed to be such for all practical purposes. To find the depth of the meters, therefore, for a given angle and a given length of cable paid out, the diagram in Plate 13B (p. 269) was constructed upon cross section paper. The curve is a parabola, plotted with its vertex up the x-axis, and the points of tangency marked for tangents making 25, 30, 35 degrees, etc., with the horizontal. The lengths of the curve from the vertex are also marked. Scales of vertical depths and horizontal distances are placed in convenient positions upon diagram. The assumption upon which this computation is based is that for a given slope of the parabola, the ratio between the curve length and the semi-chord through the point of tangency is constant. To use the diagram, suppose that the cyclometer on the winch reads 73.5 feet of cable out, and its angle with horizontal is 60 degrees. Locating the point of tangency for an angle of 60 degrees, we find the length

of curve to that point is 30.5, and the vertical ordinate is 28.8. Setting a slide rule to the ratio 28.8 : 30.5 and solving the proportion with 73.5, the recorded length of line as the third term, we obtain 69.4 as the depth of the meters.

No great refinement is claimed for this method, and it would probably not stand the test of a rigid mathematical analysis, but it is at least more accurate than the ordinary method of considering only the slope of the line, and in testing its accuracy by lowering the meters to bottom in localities previously sounded at slack water, the results agreed uniformly within 1 per cent or less, whereas, for

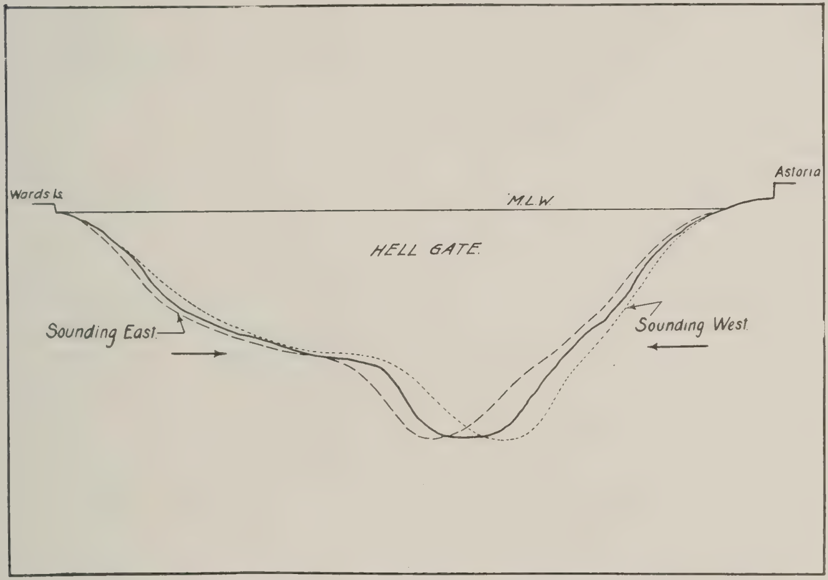


Fig. 3b (p. 270). Cross sections of Hell Gate at Negro Bluff. Horizontal scale, 1 inch equal 233.6 feet; vertical scale, 1 inch equal 93.4 feet.

the location of the meters, an accuracy of even 2 per cent would have been sufficient.

The horizontal scale on the diagram may be used in the same manner, by a proportion, to determine the horizontal displacement of the meters. This is not excessive, and, considering the straight run, no correction was deemed necessary, but if desired, the observer may, when this amount has been determined, place himself an equal distance aft of the point of suspension when he will be above the meters and may bring his new position upon range.

The procedure of taking observations was as follows:

The chief of party, equipped with a sextant, stationed himself

just outside the pilot house of the tug, a table tacked up near at hand gave the sextant angles computed for 50-foot intervals across the river. The sextant was set at the required angle for a given distance out, and the tug gradually approached the range, steering so as to arrive on line at the proper distance from shore, the meters being paid out as the line was approached. A computer was stationed in the pilot house with a slide rule, the parabolic curve depth diagram mentioned above, and a cross section chart of the meter section, from which, knowing the distance out, the depth could be picked off. As the measurement was always made at a predetermined station, and observations were made as nearly as possible at the 0.2, 0.4, 0.6, and 0.8 depths, it was a simple matter to find the depth at which the meters should be placed. (Tidal height was read and signalled from shore at the commencement of each set of observations.)

One man, equipped with a clinometer, read the slope of the suspending cable as the tug approached the line. From the required depth and this angle, the computer, with the diagram and slide rule, obtained the required reading of the winch cyclometer to place the meters at the desired depth. If the slope changed as the meters were raised or lowered, a second computation was made, taking but a second or two. An exact location at the 0.2, 0.4, etc., points was not imperative.

The speed was gradually reduced until finally the tug hung balanced in the current on range, with the meters at approximately the required depth. Then the three observers, each with a stop-watch and a telephone head receiver connected with one of the meters, observed the time for a stated number of revolutions, usually ten. These were called out in sequence to the recorder, who placed them in his note-book. The depth recorded was that computed from the cyclometer reading and the slope of the cable after the meters were in position, and *not* that at which the meters should theoretically be placed. This distance, however, between the actual depth and the theoretical points was in most cases negligible.

If the tug could be held stationary for a long enough period, the four depths at one station would be observed in succession, taking about five minutes, but if not, the approach was made as before at the same distance out, the meters at the new depth.

In this manner the stream was crossed, using, say, the 100-foot stations, then returning to the starting point and crossing again in

the same direction, observing at the 50-foot points. The two trips across consumed about one hour, giving a complete observation at each interval of 50 feet within that period.

Manifestly, a continuous observation for three days, such as was taken in Little Hell Gate, Harlem Kills and the Harlem River, was impracticable, so a set of observations extended only through one daylight tide, about twelve and one-half hours, and the varying conditions were obtained by taking one set on a spring, or large range tide, the other on a neap or small range tide.

The methods of reduction employed differ somewhat from those used in the minor channels, greater accuracy being desirable on account of the importance of this channel in relation to the whole problem. The discharge through this channel is equal to about 98 per cent of the total East River mean flow and is over twenty times the combined flow of Harlem Kills, Harlem River, and Little Hell Gate.

As a first step, the mean velocity in a vertical was determined by drawing and integrating the vertical velocity curve as fixed by the observations at 0.2, 0.4, 0.6, and 0.8 the depth. The mean abscissa to this curve was checked roughly by comparison with the 0.6 depth velocity, the mean of the 0.2 and 0.8 depth velocities and of the

$$\frac{0.2+2x\ (0.6)+0.8}{4}$$

all standard methods for computing mean velocity in a vertical. In practically all cases the agreement of these values was excellent, except near slack water, where cross currents always rendered the readings of uncertain accuracy.

Next, all velocities recorded for any one station were grouped together, and the velocity curve for that station during the period of observation was plotted on a time ordinate as explained for Little Hell Gate. For instance, at station 5+50 feet on the observation of August 4, 1911, the following observations were taken, the times of observation being the numerators and the mean velocities of the station being the denominators:

$$\frac{5{:}35}{0.0}\text{ (slack) }\frac{6{:}02}{1.94},\frac{7{:}50}{4.57},\frac{9{:}43}{4.40},\frac{11{:}24}{0.82},\frac{11{:}45}{0.0}\text{ (slack) }\frac{12{:}30}{-3.24},\frac{13{:}22}{-5.31},$$
$$\frac{13{:}52}{-5.59},\frac{16{:}11}{-4.37},\frac{17{:}38}{-1.61},\frac{17{:}55}{0.0}\text{ (slack).}$$

Similar local velocity curves are constructed for each station across the river, representing the simultaneous movement of the water in parallel vertical planes.

These curves are then divided into hourly periods, the odd intervals near slack being considered as separate periods. Obviously, these periods are the same for each curve. Choosing a certain period, say 8:00 to 9:00, we now draw upon an axis representing the width of the river a horizontal velocity curve, plotting at each station the ordinate to the local velocity curve of that station at the middle of the period.

Similarly, horizontal curves are drawn for each hourly period, and the discharge rates are computed directly from these curves by the well-known formula:

$$D=\left(\frac{V_1+V_2}{2}+\frac{d_1+d_2}{2}\right)\times L$$

where V_1 and V_2 are velocities at adjacent sections, d_1 and d_2 are depths at these same points, and L is the distance between the sections. The sum of these partial discharges for the entire width of the river is the total rate of discharge of the stream at the hour taken.

As explained for Little Hell Gate, there is now drawn upon a horizontal axis the marigram or tidal curve of the meter section (off Negro Point) during the period of the observation, together with the area curve. On a second horizontal axis, but on the same time ordinates, is plotted the discharge curve, using the rates found as above for the middle of each hourly period. Dividing the discharge at any time by the area at the same moment, gives the mean velocity, the curve of which is plotted on the same axis as the discharge curve.

Having simultaneous records of the tidal heights at each end of Hell Gate and the discharge curve as obtained above, a discharge diagram on logarithmic paper can be prepared for the north and south flow in Hell Gate, as explained for Little Hell Gate.

The averaged field observations taken at the various points indicated that the maximum discharge and velocity through Hell Gate on the northbound tide occurred at about the seventh lunar hour, and at almost the same time for the Harlem River and the other connected channels. This hour is about three and one-half solar hours after slack water in Hell Gate, and the tide is then considerably below its half height.

On the southbound tide the maximum discharge and velocity occurred at about the twelfth or zero lunar hour, or about two and one-half solar hours after slack water in Hell Gate, the tide being then considerably above half height. On account of this larger channel area at the maximum of the southerly flow, a less velocity was found than at the maximum of the northerly flow.

The discharge used in investigating the effects of diverting part of the northbound flow was 287,000 cubic feet per second through Hell Gate, deduced from the observations, as the maximum at the seventh lunar hour for a tide of mean range. For the corresponding southbound flow the discharge was taken as 282,000 cubic feet per second, occurring at the twelfth or zero lunar hour. The local slopes were obtained by observations of tidal heights at various points from 24th Street to North Brother Island, and the discharges in the connected channels and in the Harlem River were similarly deduced from the local measurements before described. Taking these maximum discharges, investigation was then made as to the reduction in flow which would result in the main Hell Gate channel from a diversion of part of its discharge. Necessarily, all results were approximate. On the northbound tide, for example, such a diversion would reduce the surface elevation at the south entrance and thus lessen the hydraulic head. A reduction of this elevation would also affect the elevation at the north end, but to an extent which could not be foretold. On the other hand, the flow abstracted from Hell Gate at its south end would be brought in again at its north end, traveling via Little Hell Gate and Harlem Kills, and thus would tend to restore the original elevations at the north end.

Again, part of the reef off Hallets Point would be excavated to a depth of 35 feet, adding about 5,700 square feet or about 12 per cent to the area of discharge. This would tend to increase the flow into Hell Gate, and the proposed excavation of Middle Ground off Lawrence Point, now a cause of strong eddies, would tend to facilitate its escape. The intervening areas, on the other hand, would be unchanged, and this would offset to some extent the tendency towards the increase of flow.

EFFECTS OF DIVERSION OF FLOW FROM HELL GATE.

For investigating the effect of transferring part of the Hell Gate flow to Little Hell Gate and Harlem Kills, the northbound tide was first taken, as this appeared to have greater velocities than the

southbound flow. Taking as a basis the discharge and the slope existing at the seventh lunar hour between 89th Street (Horns Hook) and Lawrence Point (a discharge of 287,000 cubic feet per second and slope of 1.2 feet being used as the mean of several observations) the changes in the discharge were computed corresponding to the lowering of the water surface (at 89th Street) which would result from the enlargement of Little Hell Gate and Harlem Kills. The enlarged channels were assumed to be as follows:

Little Hell Gate, 600 feet wide and 24 feet deep; Harlem Kills, 480 feet wide and 24 feet deep.

These sizes were assumed for the purposes of general computation only, and with a view to providing maximum areas. It must be remembered that they are not given as being necessarily the sizes desirable, and that the actual areas to be adopted, which might be smaller than those assumed, can only be determined after further investigations both in the office and in the field.

From the observations (which, however, were not sufficiently close to place the question beyond doubt) the surface of the water on the northbound tide, while passing through the basin between Astoria and the Manhattan shore, appeared to lose much of its slope. This change was caused by the width of channel and by the reefs off Hallets Point and in Little Hell Gate and Harlem Kills under the assumption that the change of slope caused by diverting some of the northbound Hell Gate discharge would be to some extent absorbed by this pool, and would not greatly affect the slopes past Blackwells Island.

On the basis of the foregoing, it was found that a lowering of the surface at 89th Street of about 0.55 foot, on the northbound tide at the seventh lunar hour, would divert about 80,000 cubic feet per second, or about 28 per cent of the flow now passing through Hell Gate, and would about utilize the full areas of the new channels of Little Hell Gate and Harlem Kills. The entire flow down the Harlem River from the Hudson would be passed through Harlem Kills. Neglecting the accompanying minor or uncertain effects before alluded to, such a diversion would have the following results:

Approximate reduction of mean velocity past Negro Bluff, 1.7 feet per second or 1.1 miles per hour.

(The present mean velocity is 5.95 feet per second, or 4.0 miles

per hour; the new would be 4.25 feet per second, or 2.9 miles per hour, a reduction of 28½ per cent.)

New mean velocity in the Harlem River north past Wards Island, 4½ feet per second or 3.1 miles per hour.

(This would have to be accompanied by some erosion of the present channel in order to pass the flow. The present velocity is negligible.)

New mean velocity in Little Hell Gate, 4.4 feet per second or 3.0 miles per hour.

New mean velocity in Harlem River north past Randalls Island, 2.33 feet per second or 1½ miles per hour.

(This would have to be accompanied by some erosion of the present channel. The present velocity is negligible.)

New mean velocity in Harlem Kills, 2.9 feet per second or 2.0 miles per hour.

There would also be an increase in the velocity and discharge of the southbound flow in the Harlem River north of Willis Avenue, which would probably not exceed 20 per cent, as the friction in this channel is very large.

For the southbound tide through Hell Gate, the stage at the hour of maximum flow is about 5 feet higher at Lawrence Point than at the hour of maximum flow for the northbound tide, and the discharge areas of the new channels would be correspondingly increased. The fall between this point and 89th Street is then about 1.3 feet as compared with 1.2 feet for the northbound tide.

The general effect on the southbound flow of opening Little Hell Gate and Harlem Kills would therefore be to pour a large additional volume of water into the basin around Mill Rock, raising the water surface there and reducing the present slope and velocities through Hell Gate. This would be offset in part by the increase in the discharges past Blackwells Island resulting from the raising of the level around Mill Rock, and the final resulting balance would give proportionate velocities not widely different from those obtained for the northbound tide.

Effect of Storms on Tidal Levels and Minor Irregularities in Tidal Curves

BY

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The reduction of marine soundings is often based on the assumption that in the geographical area covered by a survey, including therein the standard tide gauge, the surface of the ocean is a plane, the water surface at all points rising and falling simultaneously with that at the standard tide gauge. In long narrow tidal estuaries this assumption is so evidently far from true that a time allowance is made to partly eliminate the lag or precedence of the rise or fall of the water surface where the soundings are made, as compared with that at the tide gauge. Soundings in the offing, well outside the general coast line, with a tide gauge near the shore but to the seaward of the beach, are often thought to need no such time allowance. The object of this paper is to place on record some accurately observed and recorded data which indicate that measurable error in reducing soundings actually results from such assumptions; to show that the surface of the ocean as much as 3 miles from shore is not a plane, or simple ellipsoidal surface, but presents local elevations and depressions due to tidal flow, to the form of the bottom, to momentum causing water to flow uphill over submerged sandbars, to local differences in specific gravity of the water off the mouths of fresh-water rivers of large discharge; and that differences of barometric pressure cause similar elevations and depressions, large as well as small. It is logical to take up pronounced phenomena first, and later to investigate those of similar cause but of lesser magnitude.

STORM TIDES.

The hurricane which devastated the South Atlantic coast on August 26-27, 1893, gave an opportunity to observe the extreme

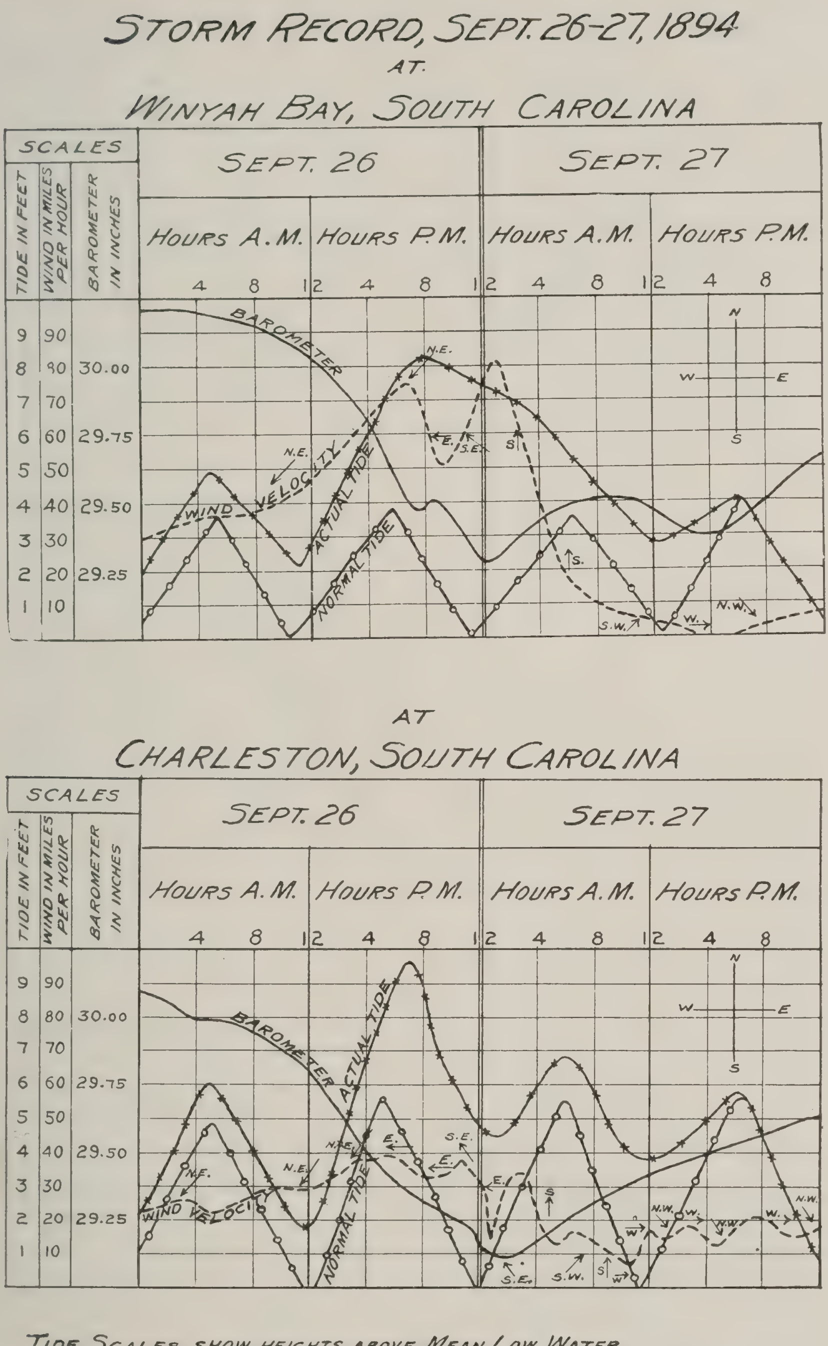


Fig. 1. (See page 282.)

heights of high water at many points near Charleston, S. C.; these points were subsequently connected by accurate levels. The following table gives the extreme elevations in feet above mean low water reached by the surface of still water in the interior of houses, etc., at the time of this storm.

	Above M. L. W.
In four different houses on the east waterfront of the city	12.65-12.97
In five different houses in the interior of the city	11.21-11.31
In one house on the extreme western water front	11.22
In two houses on the eastern side of the city, but a couple of blocks from the wharves	11.17
In a house on Sullivans Island, a quarter of a mile west of Fort Moultrie	11.25
In a house on back beach of Sullivans Island	11.20
In a house 1/8 mile east of Fort Moultrie	11.03
In a house nearby, but surrounded by low sand hills	11.80
In a magazine at Fort Sumter	11.46
Mean high water level at Charleston is	5.08

The above record shows that during this storm local levels exceeded normal by 7.89 feet and varied among themselves by 1.94 feet, due to their location with reference to the direction of approach of the water driven on shore by wind and wave, while the surface of the ocean off Charleston was locally raised something like 6 feet above normal high tide level. This storm was the worst known up to the present date at Charleston, but its tidal effects were less disastrous at Winyah Bay, only 50 miles to the northward, than a storm which occurred a year later, September 25-27, 1894. In the latter storm the tide at Charleston was only 9.7 feet above mean low water, at a time when the predicted normal tide would have been 5.6 feet. The latter storm was marked by the passage nearly over Winyah Bay of the center of the storm, as shown by general contours on the weather map, although the barometer actually fell nearly two-tenths lower at Charleston; the 1893 storm center passed almost directly over Charleston.

Of this second storm, Fig. 1 (p. 281) shows the great difference in tidal effects at the two localities. At Winyah Bay it is seen that diminution of the barometric pressure so held up the tide that at the time for low water the surface was less than 1 foot below its highest elevation. This super-elevation of water corresponded with a barometer fall of nearly an inch in twenty-four hours. With such an enormous elevation of the level of the ocean, due solely to atmospheric causes, it is evident, reasoning by analogy, that in ordinary weather local whirls and eddies in the air may correspond to smaller mammæ on the water surface, sufficient in magnitude to vitiate the mathematical exactness of tide reductions applied to

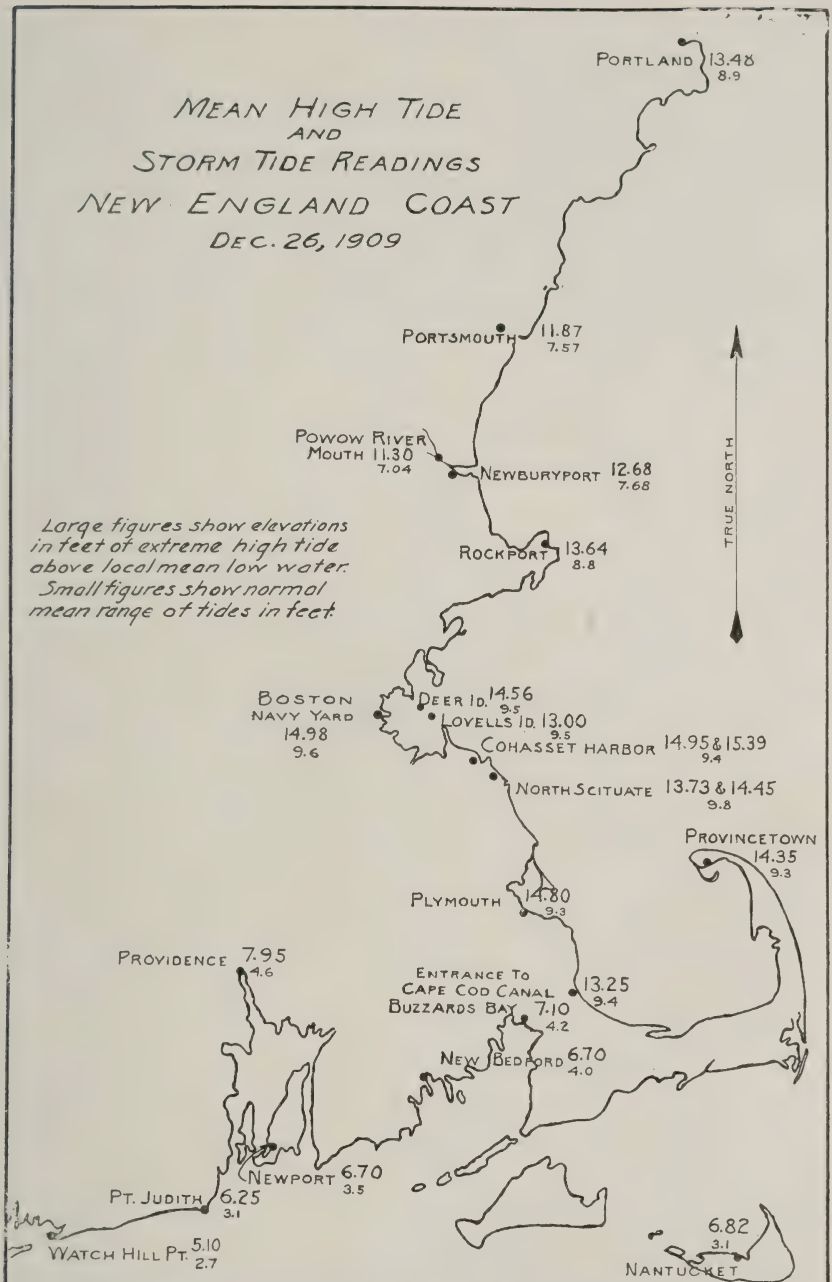


Fig. 2. (See page 284.)

observed soundings. Abnormal humps and hollows in the record curves drawn by automatic tide-gauges are visible evidence that such vagaries of surface reach measurable magnitudes, certainly one or two-tenths of a foot, not infrequently.

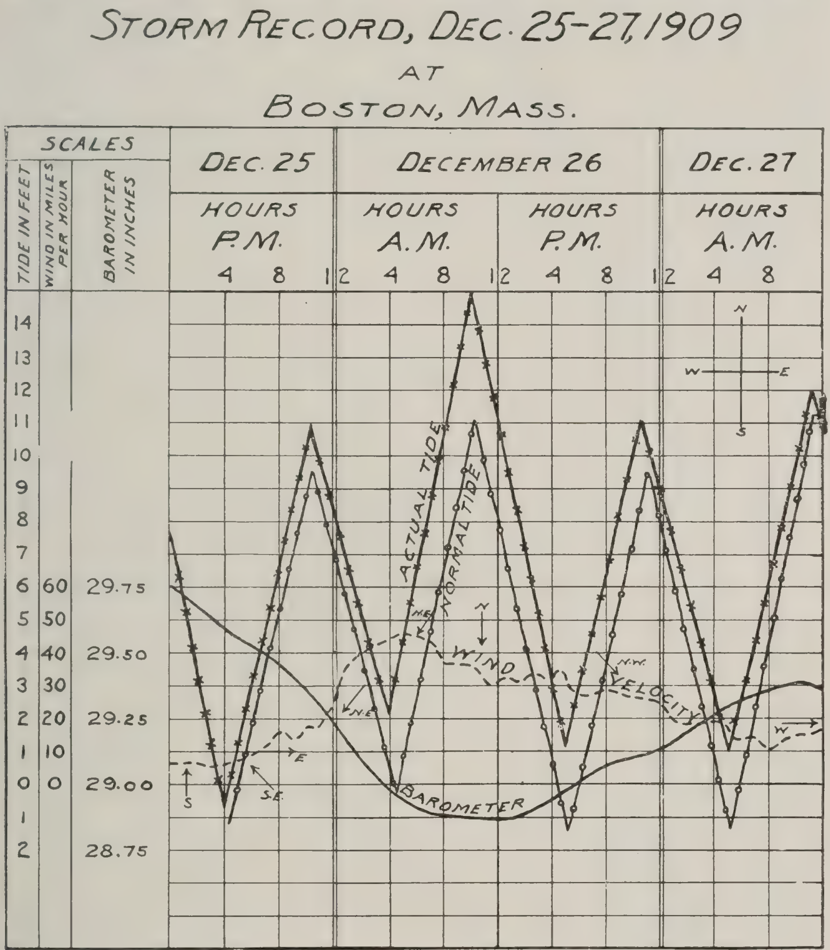
In 1909, on December 26, a storm of unusual severity occurred on the North Atlantic Coast, and advantage of the opportunity was taken by Col. Edward Burr to tabulate extreme high water marks along the coast from Portland, Me., to Block Island. The accompanying sketch map (Fig. 2, p. 283) shows the location of tide observations and the elevation of the storm tide at its extreme height, referred in all cases to local mean low water, and the predicted normal level of high tide on that date. This map illustrates the effect of the general form of the shore line with reference to the direction of wind; and detailed study of the complete records in the Boston office, where great numbers of readings are listed, shows that small local features resulted in disproportionate vertical distortion of the high-water surface—for example, a difference of 1.56 feet between high-water level at Deer Island and Lovells Island, Boston Harbor, only about a mile apart.

The Boston storm was one in which the low area was of great extent, while the South Atlantic hurricanes above described had more pronouncedly accentuated central depressions, like the vortices of tornadoes, in addition to the widely extended general low barometer usually marking a violent storm. This difference accounts for the local damage at Charleston and comparative immunity of Winyah Bay in 1893, and the reverse in 1894, though these localities were separated by only about 50 miles; while the low lands along the whole New England coast from Portland to Cape Cod were submerged by the extremely high tide resulting from the storm of 1909, in which the barometer fell about as far and as rapidly (1 inch in twenty-four hours) as in the case of the earlier storms on the Carolina coast. (See Fig. 3 for full barometric data.)

The exaggerated effects produced on the water surface by extreme storm conditions having been shown, the next step is to gather together data to show whether under normal every-day conditions the tidal rise and fall over such areas as are usually covered by harbor surveys may safely be assumed as approximately uniform.

A special study of tides, made for Colonel Burr in 1908 in Boston Harbor by Lieutenant Park, is instructive. The object was to obtain by simultaneous tidal readings a number of permanent reference points for tide-gauges. Staff gauges were read from about

three-quarters of an hour before high tide to three-quarters of an hour after. Watches were set and compared with great care, and the readings were taken about once a minute as accurately as possible. Figs. 4 and 5 show the results of readings on September



0 OF TIDE SCALE IS MEAN LOW WATER

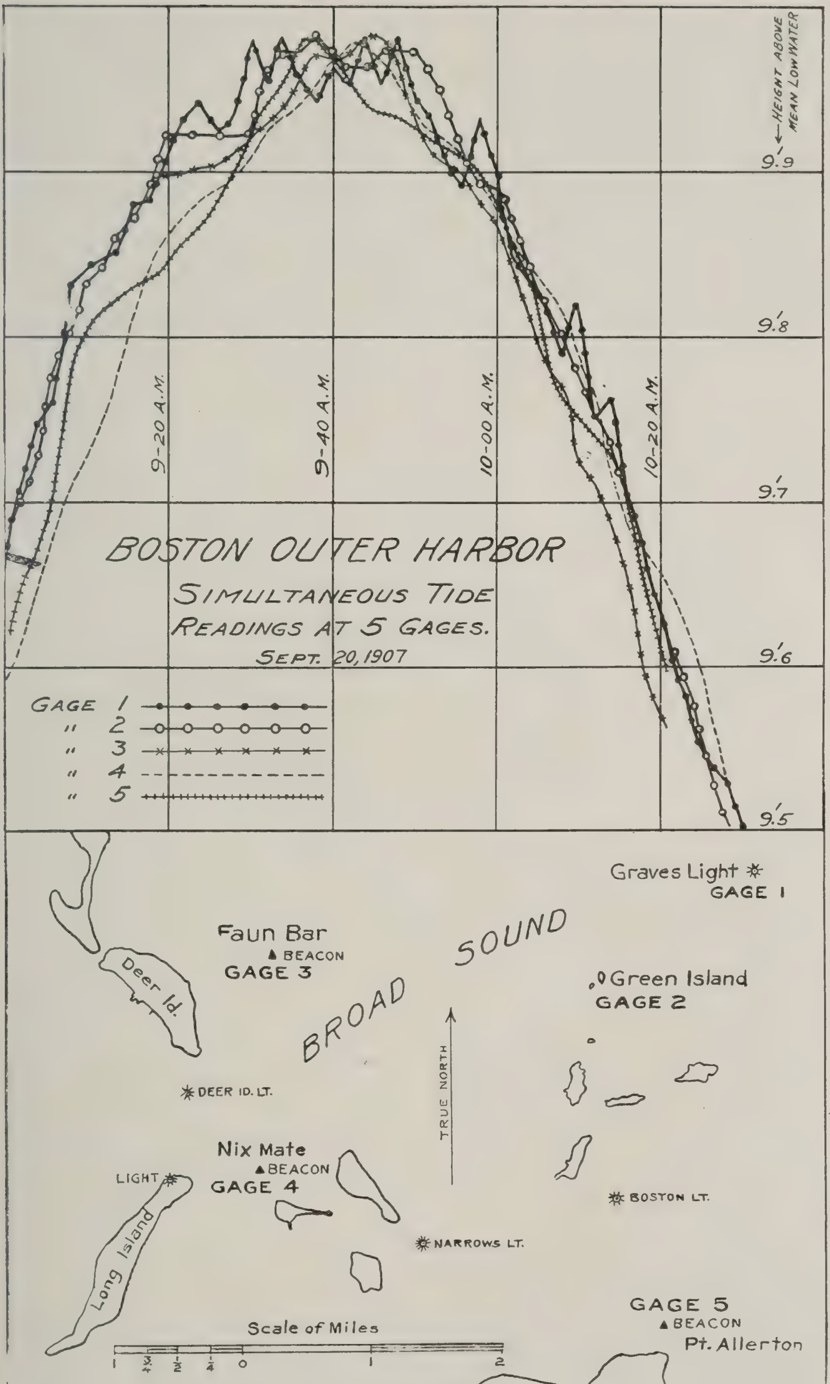
Fig. 3.

20 and September 21; the former shows numerous irregularities in all the curves, the latter more regular curves. The small map at the bottom of each plate shows the relative positions of the gauges. Vertical differences between gauges, amounting to as much as half a tenth of a foot are apparent, even though the readings all lie

within less than half a foot of high-water slack. At the outer gauge the shape of the tide curve is sharper on the 21st and flatter on the 20th than that of any other gauge. The difference in time between equal readings on different gauges varies as much as ten minutes, and gauge No. 4, which shows the longest period of readings above reference 9.8, on September 21, shows the shortest period of readings above that level on the 20th, although the two tides were of almost exactly equal height. If the observations had continued till steep tidal slopes had become established, doubtless much greater differences would have been shown. At the outer gauge on both days there were well-defined periods of rising followed by falling and succeeded by rising, which were real phenomena shown by numerous consecutive observations. None of the curves are the smooth, regular sinusoids called for by theory.

Fig. 6 shows records from two automatic gauges in Newport Harbor; differences as great as three-tenths of a foot are shown between these gauges only half a mile apart. The accuracy with which the irregularities in one curve have shown day after day on the other indicated that these oscillations must be real local phenomena, not accidents due to poorly running mechanism of the gauges; but they were so pronounced and unusual that I had readings made, taken by careful observers of the staff gauges, and they verified the automatic curves. These gauges were used for reducing soundings on a rock-removal contract, where tenths of a foot meant dollars; the ledge was equidistant from the two gauges, and it was no simple matter to determine what gauge to use. The greatest variation shown on Fig. 6 (p. 290) is close to slack water, which is not usual, but is like the Graves gauge record on Figs. 4 and 5 (pp. 287 and 289). I have personally known cases where, at the strength of the tide, two staff gauges on piles, less than ½ mile apart and both far from the nearest shore, differed in their simultaneous readings by half a foot, though at slack water of both high and low tide their readings were identical.

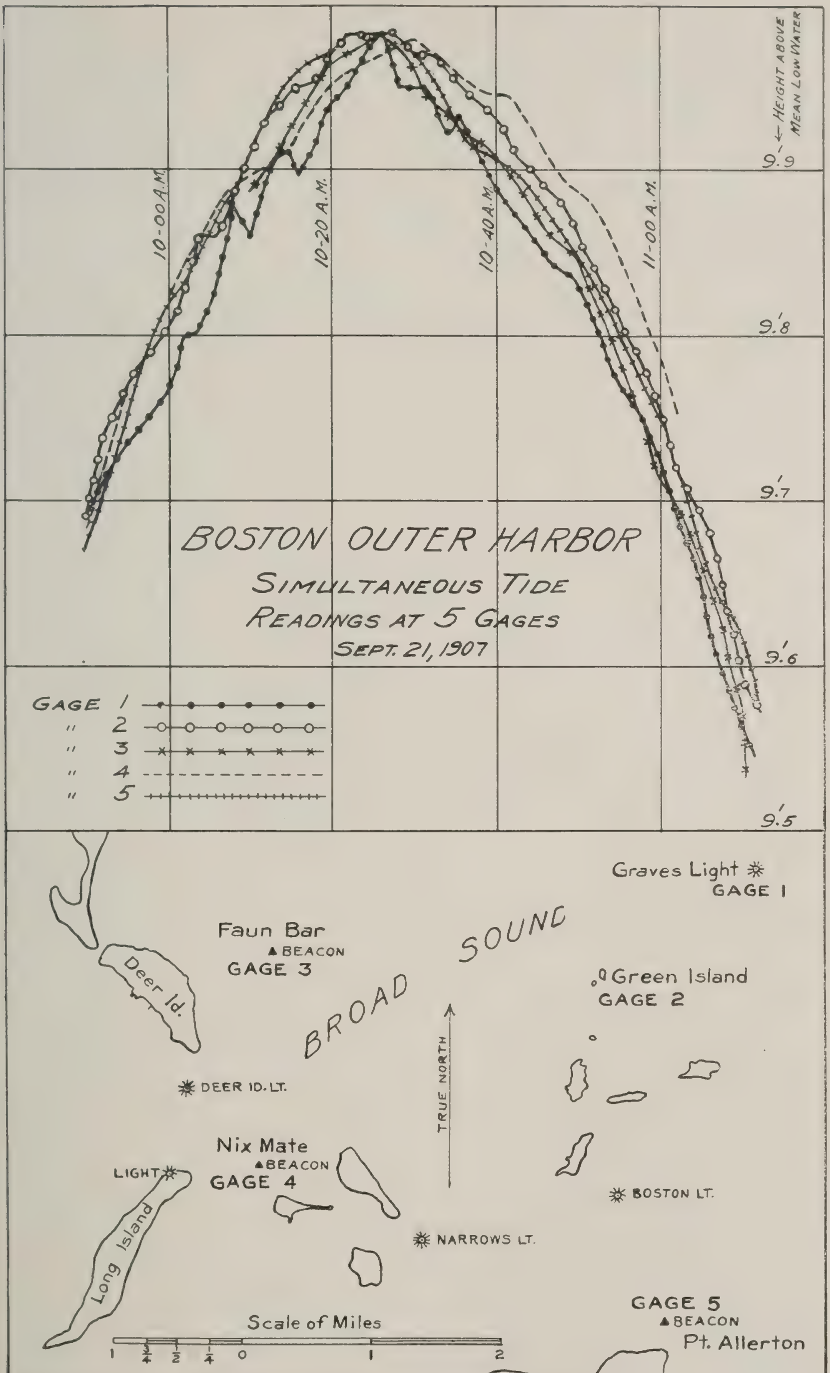
With stretch of lead-line, bow in the line where the current is strong, lack of verticality, penetration of the lead into soft bottom, errors in location, unknown tidal slopes, wash of surface chop up and down on the line, long ground-swells, raising the surface of the water for seconds at a time, and motion of the boat, marine hydrog-



BOSTON OUTER HARBOR
SIMULTANEOUS TIDE
READINGS AT 5 GAGES.
SEPT. 20, 1907
HEIGHT ABOVE MEAN LOW WATER
9.9
9.8
9.7
9.6
9.5
9-20 A.M.
9-40 A.M.
10-00 A.M.
10-20 A.M.
GAGE 1
" 2
" 3
" 4
" 5
Graves Light
GAGE 1
Faun Bar
BEACON
GAGE 3
BROAD SOUND
Green Island
GAGE 2
Deer Id.
DEER ID. LT.
TRUE NORTH
Nix Mate
BEACON
GAGE 4
LIGHT
Long Island
BOSTON LT.
NARROWS LT.
GAGE 5
BEACON
Pt. Allerton
Scale of Miles
1 3/4 1/2 1/4 0 1 2

raphy is not the exact science it is sometimes thought to be. Even with poles in place of lead-lines, it is extremely difficult to make lines of soundings which cross show identical depths at the points of intersection. The duty of the hydrographer is not only to eliminate all known causes of error, but in addition to realize that for large areas his map will average close to the truth, but that individual soundings and even large groups of soundings taken at about the same time may be several tenths in error in spite of all he can do. The best rule I have been able to find is, Always to check lines of soundings by others on ranges crossing the first set. Where one line uniformly shows soundings deeper than those it crosses, reject that line and run it over again. I have known cases where a whole day's soundings or a whole half-day's soundings, accurately taken, apparently under the best conditions, failed to check cross lines within half a foot; a re-running resulted in entire agreement with the same cross lines within a tenth or two. The trouble was unquestionably an abnormal tidal slope between the area sounded and the tide-gauge used to reduce the soundings.

At Brunswick, Ga., in 1895, it became necessary to determine with great accuracy the elevation of mean high water on the bar, 4 miles from shore with no islands intervening between the gauge and the beach. Mean low water was accurately known at an automatic tide-gauge station at the nearest point of the shore. A self-registering tide-gauge station was built on the bar in about 18 feet of water, and a self-registering tide-gauge was operated there by a clock, electrically wound once an hour. It was necessary to have the gauge run as long as two weeks at a time, when storms made it impossible for the gauge tender to land at the station. Every known refinement was applied to the maintenance and observation of the gauges and to the reduction of the records. Contrary to preconceived ideas, a well marked, quite uniform difference of range was found, as shown by the following table giving by half lunations the mean range in feet at the inner gauge, at the outer gauge, and the difference between the two.


← HEIGHT ABOVE MEAN LOW WATER
9.9
9.8
9.7
9.6
9.5
10-00 A.M.
10-20 A.M.
10-40 A.M.
11-00 A.M.
BOSTON OUTER HARBOR
SIMULTANEOUS TIDE READINGS AT 5 GAGES
SEPT. 21, 1907
GAGE 1
" 2
" 3
" 4
" 5
Graves Light
GAGE 1
Faun Bar
BEACON
GAGE 3
BROAD SOUND
Green Island
GAGE 2
Deer Id.
DEER ID. LT.
TRUE NORTH
Nix Mate
BEACON
GAGE 4
LIGHT
Long Island
BOSTON LT.
NARROWS LT.
GAGE 5
BEACON
Pt. Allerton
Scale of Miles
1 3/4 1/2 1/4 0 1 2

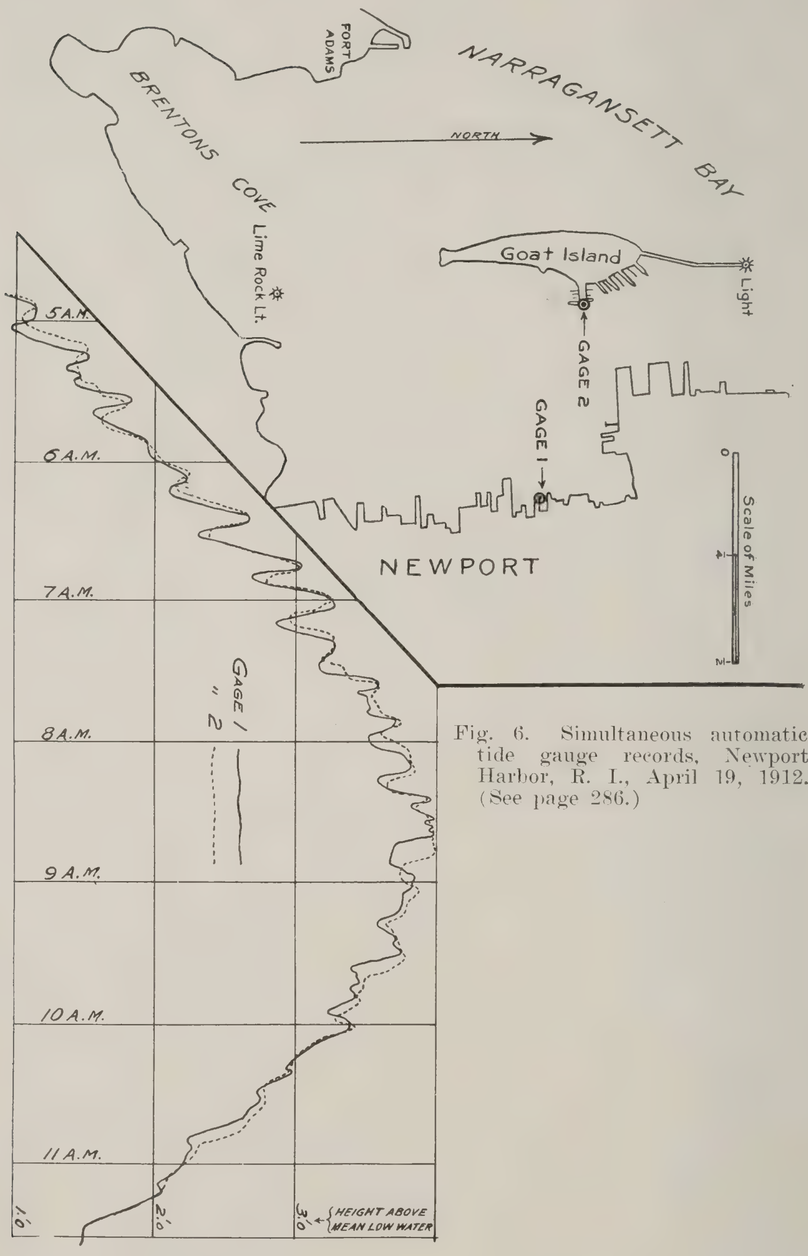


Fig. 6. Simultaneous automatic tide gauge records, Newport Harbor, R. I., April 19, 1912. (See page 286.)

Table of Means of Tidal Ranges at Brunswick, Georgia, March to November, 1895, by half lunations.

Outer Gauge.	Inner Gauge.	Difference.
Feet.	*Feet.*	*Feet.*
6.047	6.243	0.196
7.148	7.295	0.147
5.989	6.158	0.169
7.020	7.189	0.169
5.921	6.080	0.159
6.920	7.122	0.202
6.245	6.427	0.182
6.646	6.851	0.205
6.768	6.957	0.189
6.224	6.435	0.211
6.979	7.182	0.203
5.983	6.180	0.197
6.992	7.127	0.135
5.491	5.665	0.174
7.067	7.205	0.138
6.496	6.674	0.178
5.524	5.691	0.167
5.987	6.145	0.158
Mean 6.414	Mean 6.590	Mean 0.176

The New Italian Field Gun*

TRANSLATED BY

Capt. W. G. CAPLES
Corps of Engineers

Since the appearance of the French 3-inch gun, all the great powers have gradually changed their artillery matériel. Everywhere, rapidity of fire has been the object sought by the constructors who have solved the problem with more or less success or good luck until all modern guns can now be considered as about the same. In general, they differ in war only in the manner in which they are kept supplied and in their tactical employment. From this point of view, the example of the present Balkan war is very suggestive.

On the other hand, it seems that in rapidity and precision of fire and its destructive effect, a maximum has been attained that has not been exceeded for several years. That does not say that further progress along these lines is impossible, but rather that research has been along others. It is quite certain, for example, that the increase of the field of action of the gun and its more nearly perfect adaptation to the multitudinous needs of modern tactics are very desirable improvements to which no artilleryman can remain indifferent.

It is along these lines that Colonel Deport, the inventor of the 3-inch gun, has directed his studies. The matériel that he has de-

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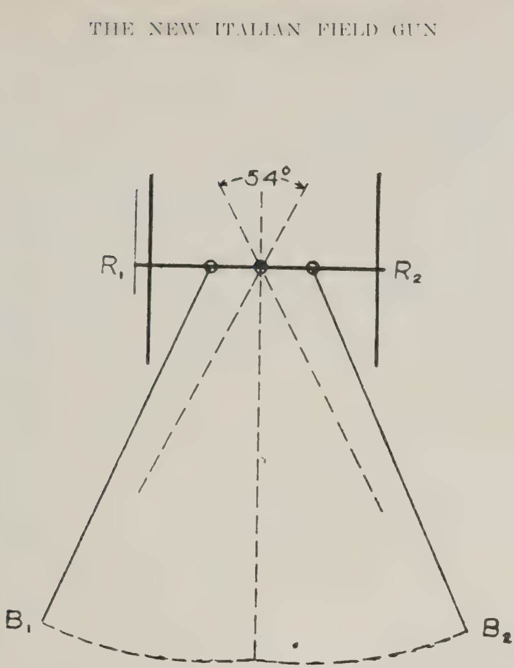
*Translated from *Militaire Suisse* for January, 1913. One of these guns has been brought to this country by the Ordnance Department and tested by the Field Artillery Board with a view to seeing whether any of the features of the carriage were worthy of being adopted for United States service. It is understood that there are some tests yet to be carried out, but that so far the reports are favorable to the type of carriage invented by Colonel Deport. The Deport gun has also been adopted for use by the United States Marine Corps for the advanced base of an expeditionary force. For further information, and particularly for Colonel Deport's own account of his gun, see translation of *Revue D'Artillerie* of June, 1911, published in the *Field Artillery Journal*, January-March issue, 1912, also translation from the German of an article on "Italian Field Guns," published in October-December, 1912, issue of same journal.—ED.



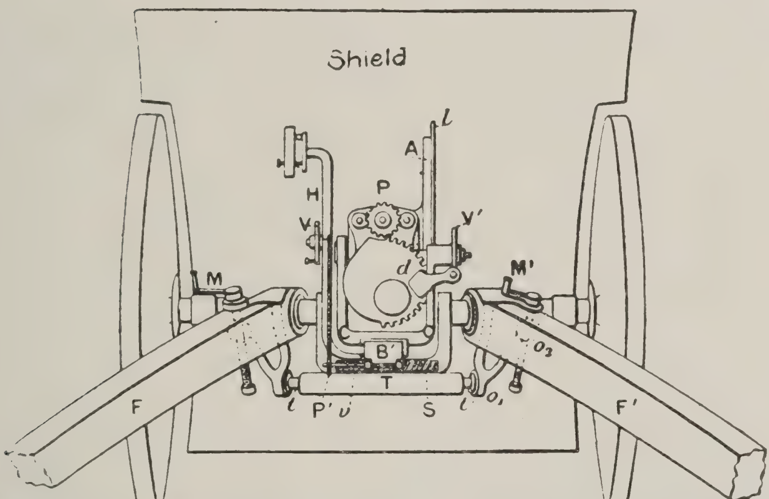


Fig. 1 (upper). Sketch plan of Deport gun, showing field of fire and relative positions of the wheels and trails. (See page 296.)
Fig. 2 (lower). Sketch of rear view of gun with principal parts lettered. (See page 296.)

signed and that has been adopted in Italy after long and exhaustive tests, presents, along these lines, some advantages that it appears interesting to demonstrate.

It would be superfluous to explain to the readers of this paper how great rapidity has been attained in the fire of modern field artillery. But, whatever may have been the means employed, this rapidity of fire has, up to the present time, always had its drawbacks; that is to say, a very narrow limitation of both horizontal and vertical field of fire, a limitation that does not fail to be very inconvenient.

Practically, the narrowness of the field of fire requires that aiming at any object whatsoever involves first the pointing of the carriage almost with the same degree of nicety required by the old-style cannon. This operation is much longer and more difficult when the gun has been previously fired and the carriage thereby has been firmly anchored in the ground. Such a procedure is so slow that it loses much of the advantage gained by the rapidity of fire possible with the gun.

The reduced field of vertical fire has consequences not less unhappy. Without attempting to enumerate them all here, it will suffice to invite attention to the difficulties met with, in mountainous country or simply by accident—where the slopes often prevent attaining a fire of much effect—to give the piece a sufficient inclination Generally, success in such instances has been obtained only by improvised means and with the double difficulties of delaying the opening of fire and interfering with its regularity.

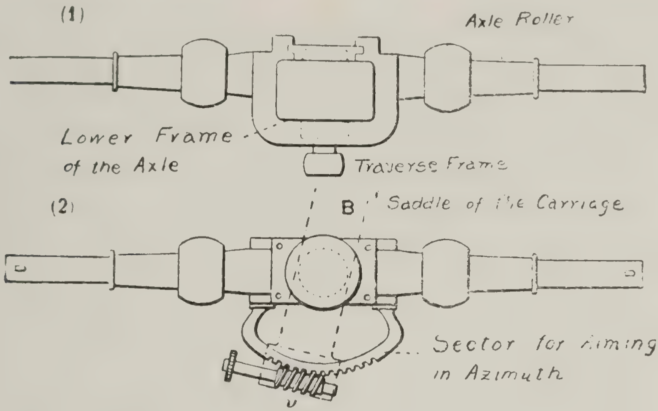
In spite of all the progress made in the last twenty years, the guns now in use are far from the ideal cannon dreamed of by General Langlois—that is, a gun that will pour shrapnel upon any point on the field of battle as easily as water is turned with a hose upon the different parts of a burning building.

But in doing away with all changes of position of the carriage for firing upon moving objects or change of objective, in considerably extending both the vertical and the horizontal fields of fire and, finally, in simplifying still more the duties of the gun pointers, it seems that Colonel Deport has approached remarkably near the ideal.

ATTAINMENT OF A LARGE FIELD OF HORIZONTAL FIRE.

To permit wide changes in orientation by moving the gun on the carriage without affecting the stability of the latter and, consequently, without deranging the pointing of the piece, it is better

to use four solid points of support upon the ground rather than the three now used. This result has been attained by substituting, for the one-piece trails now in use, two trails arranged to rotate in the axle. These two trails, joined together on the march in such a manner as to permit the usual arrangement, can open, at the instant of going into battery, so as to inclose a sector of 54 degrees.



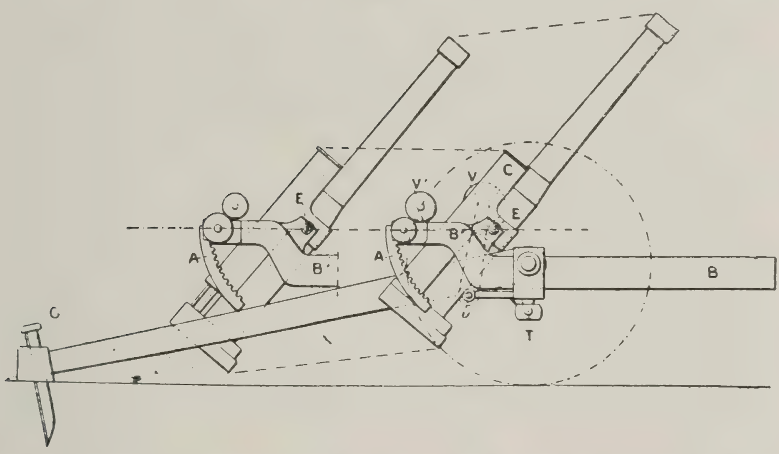


Fig. 3 (upper). 1. Recessed axle of Deport gun, shown in vertical projection. 2. Recessed axle in horizontal projection. (See page 296.)

Fig. 4 (lower). Sketch of vertical projection of gun, showing position before and after being fired at 60 degrees elevation. Note that the gun has recoiled downward, while both cradle and gun have recoiled horizontally to the rear. (See page 298.)

The ends of the two trails rest on the ground by means of two shoes forming large surfaces of support and are immobilized by two trail spades driven with a small sledge.

The wheels carry a recessed axle, cut in its center in the shape of a bracket block. At the lower bed of this frame is fixed a pivot upon which the bearing of a revolving saddle engages. At the same time that it shows the seating of the carriage on the ground, the sketch (Fig. 1, p. 293) shows the extreme directions between which the saddle—and with it the gun—can be displaced without deranging the aim.

The changes in azimuth of the revolving saddle, called the carriage saddle, are made in the following manner. At the rear post of the lower bearing of the axle is fixed a toothed arc concentric with the axis about which the saddle revolves. (Figs. 2 and 3, pp. 293 and 295.) A worm, V, fastened to latter engages in the toothed arc. This worm is actuated by a pinion which is moved by a sprocket chain that is itself actuated by another pinion. This second pinion is carried on the arm "H" forming part of the saddle and keyed to the axis of a hand-wheel "V" for pointing in azimuth. The hand-wheel "V" controls all the movements of the sprocket chain and permits turning the saddle in any desired direction. Each turn of the wheel corresponds to a deflection of 32 millièmes or 2 degrees. The pivoting system is balanced in such a manner as to render the movements very easy and rapid and to facilitate changes in orientation. It requires five or six seconds to go from one extremity of the field of fire to the other.

A Goerz panoramic glass fixed on the arm "H" serves as the means of sighting. A suitable small mechanism permits keeping the axis of suspension of the glass parallel to the axis of the gun and of automatically correcting the inclination of the axle and the drift.

This arrangement, combined with the opening trails, makes it possible to increase the breadth of the field of horizontal fire from the 6 or 7 degrees obtained with the present material to 54 degrees, without affecting the stability of the carriage and without deranging the aim.

It is clear that a rotation of the carriage about of the spades, "B," for example (Fig. 1), is impossible so long as the axis of recoil prolonged remains inside the lines B_1B_2, the spade B_2 forming an obstacle to rotation about B_1. But the derangement of the piece could still be due to another cause. The action of the recoil tends to turn the whole carriage, according to the direction in which it acts, about the line R_1B_1 or R_2B_2, going from the extremity of the spindle of the axle to the spade. But mathematical deductions,

which need not be given here, show that derangement is not to be feared so long as the axis of the saddle remains inside the angle formed by the axes of the two trails prolonged. In practice, it is possible even to go a little beyond these limits. One of the wheels may be slightly lifted at the moment of recoil but, thanks to the solid anchorage of the carriage in front and behind, it falls back

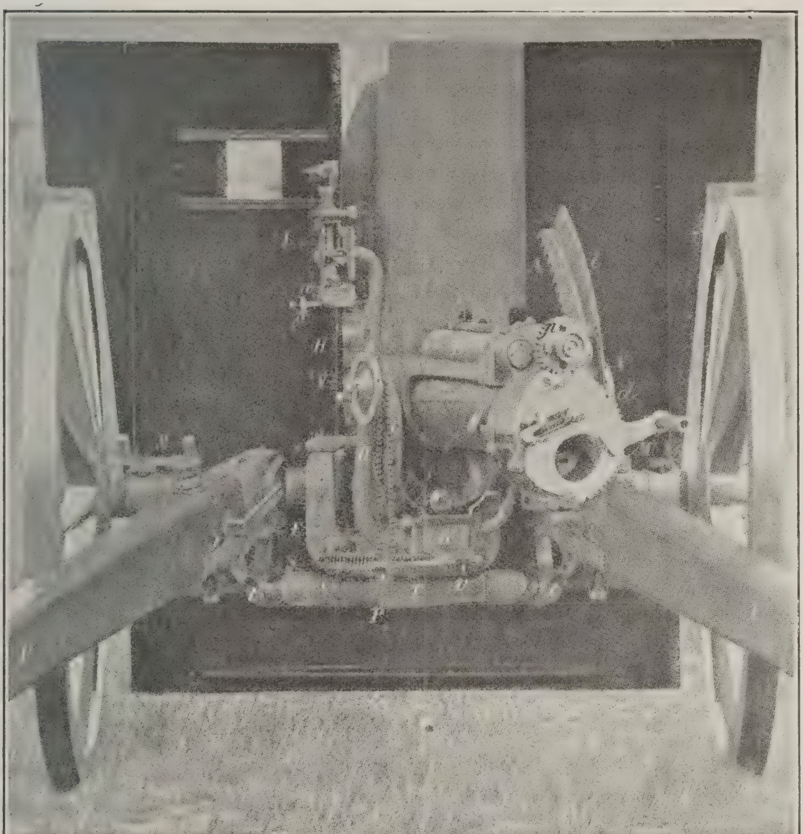


Fig. 5. Rear view of Deport gun. Breech in extreme right position.

exactly into its former position and produces no change in either inclination or direction of the piece.

LARGE FIELD OF VERTICAL FIRE.

The small height of the trunnions above the ground in modern guns makes it impossible to gain an extensive field of vertical fire

when the only device used to take up the recoil is the recoil cylinder with an axis 1.3 meters long.

In place of the single recoil cylinder, Colonel Deport has substituted:

1st. A shoe recoil check sliding on the cradle of the carriage with a run of only 1 meter.

2d. A recoil cylinder (berceau-frein) on the piece, with a run of 0.36 meter. This recoil cylinder rests on the shoe by two bearings, which permit an elevation of 50 degrees. (See Fig. 4, p. 295.)

The carriage cradle and, consequently, the shoe recoil check, on their part, can be elevated from —10 degrees to +10 degrees.

The elevation of the carriage cradle is obtained in the following manner. The lower frame of the axle is fastened to a crossbar, "T," by a toggle joint (articulation á genou). This crossbar is fastened at its ends (t and t') to the trails, one side, and to the axle on the other by means of the two aiming sectors, which are actuated by two hand-wheels, M and M′. Turning these wheels causes the crossbar, "T," to rise or fall and gives to the sill of the axle frame an oscillatory movement which is transmitted to the carriage cradle and by it to the slide. If only one hand-wheel is used the movement is only half as rapid.

On the right of the gun cradle is fixed a solid toothed arc engaging with a pinion carried by the carriage shoe (traineau d'affût); a hand-wheel, V′, controls this pinion by means of another worm gear. Turning this hand-wheel gives the axis of the gun the desired elevation with respect to the axis of the shoe (traineau).

When firing at small angles the courses of the two recoil checks are added to form a total course 1.36 meters long, insuring stability. In firing at large angles, on the contrary, where lifting of the carriage is not to be feared, the cannon recoils downward only 0.36 meters and can be easily loaded and fired. The two recoil checks adjust themselves automatically to any angle of fire without requiring any connection between them.

The field of fire thus attained ranges from —10 degrees to +60 degrees, making an amplitude of 70 degrees; four times greater than possible with the present guns.

SEMI-AUTOMATIC BREECH MECHANISM.

The adoption of a semi-automatic breech mechanism completes the improvements by relieving the gunner on the right from leaving to open the breech, thus facilitating the aiming and increasing the regularity of fire.

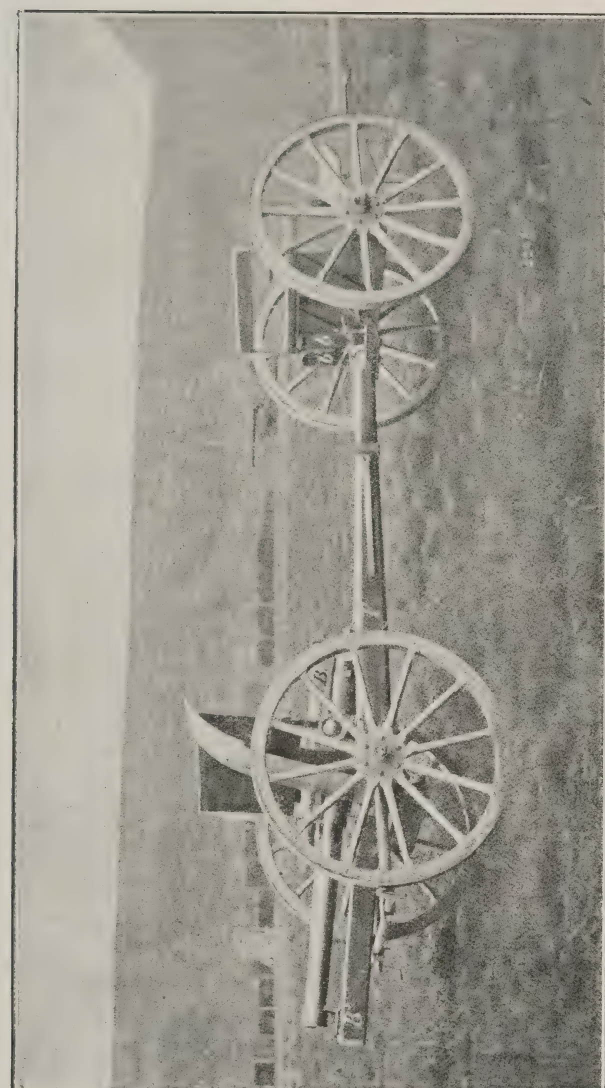


Fig. 6. The Deport gun limbered up.

The breech is of the same system, eccentric screw, as that of the 3-inch gun, well suited to being made semi-automatic.

To the circumference of the breech-block, which makes the closure, is fitted a toothing which engages a pinion, "P," fastened to the cannon and keyed to an arbor formed by a reversible quick-motion (à long pas) screw. The screw engages in a nut in a central cylinder forming part of the cradle. This nut, from the nature of its setting, can have only a simple motion of translation. When the nut is run forward or back it turns the screw and through it gives the necessary closing or opening movement to the breech.

The movement of the nut is produced by two springs which, compressed by the recoil, serve, one to open and the other to close the breech.

(The article here continues with the details of the service of the piece, interesting rather to the artilleryman and the ordnance expert than to the engineer. Briefly, the convenience and simplicity of the firing arrangements have been improved. The piece is quickly set and rapidly shifted to cover any target. Four men are required to serve the piece, one to give direction, one to give elevation, one to load, and one to pass charges to the loader. The first three are protected by the shield of the piece and the fourth by the limber.)

The piece is placed in battery at least as rapidly as with the present gun. This is not surprising, since with the wide field of horizontal fire the initial setting of the piece does not have to be so accurate. As soon as the piece is in battery, the gun crew are fully protected before the first shot is fired.

The Italian tests show that the gun travels as well as, if not better than, the present guns. As to its ruggedness, an accident happened that gives a good illustration. Due to carelessness in not setting the brakes, a gun and limber ran down and over a cliff 200 feet high. One wheel and one trail were broken, the shield was bent, the limber was stove in, and the foot of the other trail was injured. None of the mechanism was either injured on the gun or on the carriage. A few days repairs and a new trail put the piece into service again.

In comparative tests with the Schneider and the Krupp, the Deport gun proved superior and met every requirement. In one test, each piece was required to fire 500 shots in six hours. All four of the Deport guns met the test, the recoil checks acting regularly

Fig. 7. The Deport gun trained to fire at extreme angle of elevation, —60 degrees above horizontal.

and none of the parts showing any appreciable wear after the test. The Krupp guns tried to get through by removing part of the liquid from the cylinders but the pieces failed to return, sometimes by as much as 0.25 meter, causing the fronts of the cradles to swell and give way from the blast of the guns. Only one Schneider gun out of four could fire the full 500 rounds, but immediately afterward the liquid in the check overflowed.

The tests left no doubt that the Deport gun is superior to the Krupp and to the Schneider in the ease with which it is placed in battery, the execution of fire, the accuracy of fire, the resistance of the material to shock and to the march, and in the degree of perfection of the mechanical devices.

TACTICAL ADVANTAGES.

(The article treats of the subject, naturally, from an artillery view point, which, in many cases, is only of passing interest to the engineer. The Italian gun, if the reports be correct, will fill all the functions of a field gun, a howitzer, and a mortar. Where such a gun is to be used, it is possible for the engineer to locate his works so close up under a reverse slope that nothing short of a howitzer or a gun of the same type can silence the fire of the gun emplaced. If the natural slope is not steep enough to cover the gun, a little scraping of the hillside—the simplest and easiest kind of work—will give an emplacement from which the Deport gun can fire with complete immunity from anything less than a gun of the same type or of a mortar. The wide field of horizontal fire makes the problem of mutual support far simpler, as positions which formerly could not act together can now cross their fire. Concentration of fire can be obtained with an even greater dispersion of guns. Divided into small widely spaced groups, the Deport offers a hostile artillery no chance for concentrated fire. The rapidity with which a change of target can be effected makes firing against infantry frequently possible where formerly the time required for change would result in the loss of the target. At a range of 3,000 yards a gun without change of position can in six seconds run over a front of 3,000 yards. If the ranges are established in advance, infantry exposing itself even for a moment will be severely punished.

(The ability to search out rear slopes raises new problems in shelters for reserves and for communication. Evidently, the artillery need no longer stop firing when the attacking infantry most

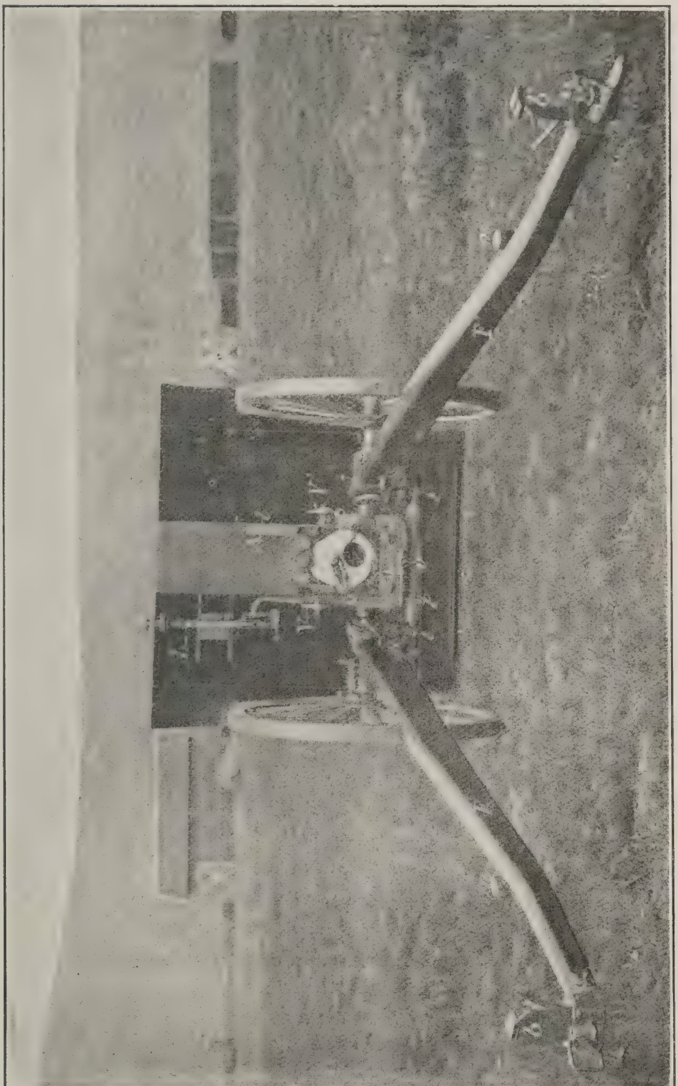


Fig. 8. The Deport gun as seen from directly in rear when in position to fire horizontally or with only a slight angle of elevation and straight to the front.

needs support, but may simply change its target slightly and prevent the arrival of reserves and turn an ordinary defeat into a panic if the defenders have to retreat over open ground, even though it be on a rear slope.

(Probably the most noticeable change to meet the action of such a gun will be in the cover provided infantry trenches. For a long time it has been almost axiomatic that an infantry trench effectively reached by artillery is untenable. In the Russo-Japanese War, artillery reached the point where it was able to search trenches quite effectively. One very competent observer expressed the opinion that a trench without headcover, unless concealed, was a detriment rather than an aid, since it showed exactly where the defenders were and gave them no real protection since it left their heads as vulnerable as in the open. This same observer holds that a soldier using natural cover is less liable to be hit than one in a visible trench without headcover. In tests made for the Board on Engineer Troops in connection with the supply of intrenching tools to infantry, it was apparent that, even under rifle fire only, the chances of killing a man lying down and of killing one with his head and shoulders exposed over a trench are about the same. Aside from other blunders, there is no doubt that the nature and position of the Russian trenches at the Yalu contributed largely to the Russian defeat. The loss of the strong Russian position in the first day's fighting at Liao-Yang was due in no small measure to two little mountain guns that were able to get into a concealed position and fire directly into the Russian trenches. Instances could be multiplied to show that artillery is increasingly able to get into the infantry trenches, and when it does get in the infantry must leave.

(The Deport gun acting as a mortar can attack a trench with unimpeachable thoroughness. Concealment, already a prime requisite of a good trench, necessarily becomes of greater importance than ever. Slopes must be so flat as to be invisible at very short ranges, crest lines must follow the natural contours even closer, and the most scrupulous care must be taken to make the parapet blend with foreground and background or disappear entirely behind a few rows of corn, a little brush or other natural concealment.

(With projectiles falling at mortar angles, mere head-cover will no longer answer for lack of invisibility. Overhead cover as well must be added to guard against shrapnel, while the already frequent traverses must be brought closer together to limit the effects

of high explosive shell. If the ammunition supply can be kept up, a gun capable of firing 500 rounds in six hours must sooner or later search a visible trench. The thickness of overhead cover will tend to increase, obstacles will take on an added importance, and the operations, like some of those on the Sha-Ho, will approach more nearly those of a siege under the influence of a gun that is at once field gun, howitzer, and mortar.

(While the dead space of indirect fire is materially lessened for a gun that can act as a mortar, the necessity of being able to use variable charges and fixed ammunition at one and the same time presents some problems for the ordnance expert before the Deport gun can reach its full value, but, like all other problems, the genius of some man will offer a solution.

(Certainly, if the Deport gun is the success that it is reported to be, the answer to the question of how to meet its offensive power is evidently its universal adoption and, in common with the Swiss reviewer, ''We see how new and fertile is the field opened by the eminent engineer and we may feel sure that in this instance, as in preceding ones, he will find many imitators who will have to follow in his path.'')

Bvt. Maj. Gen. Simon Bernard*

BY

Maj. Gen. WILLIAM H. CARTER
United States Army

Nearly a hundred years ago, when America was rankling with the humiliation of the second war with England, relieved only by the brilliant campaign of Jackson, the President advised our representative in Paris to secure the services of an eminent military engineer to supervise the fortification of our coast. Paris was filled with French officers without employment by reason of the final downfall of Napoleon at Waterloo. In looking over the field of experienced officers our representative had the good fortune to encounter Gen. Simon Bernard at a moment when he had been warned by the French minister of war that for his personal safety he, Bernard, should quit France without delay. Bernard's reputation as a military engineer was of so high an order that his services were eagerly sought by several European governments, and most flattering offers were tendered to him, all of which he declined in order to follow the example of Lafayette, Rochambeau, du Portail and other gallant Frenchmen who had gone to aid America in the Revolution. The life story of this modest and unassuming gentleman, whose services to both America and France were of the highest character, reads like a veritable romance.

Simon Bernard was born at Dôle, France, the 22d of April, 1779. He was educated at the Polytechnic School and entered the French Engineers when all Europe was an armed camp. The Napoleonic era was filled with strenuous life for the men with the colors, and the young engineer participated in many stirring campaigns, winning always the highest commendation of his superior officers. He

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*Reprinted from the *Journal of the Military Service Institution of the United States*, by special permission of both General Carter and the publishers of the journal. The picture (see frontispiece) was presented by Gen. C. de Grandfrey, of the French Army, to Maj. F. A. Mahan, Corps of Engineers, retired, and by him forwarded to the MEMOIRS.

was present and led the assault on Ivre in 1800, being at the time only 21 years of age. During the celebrated siege of Torgau, notwithstanding his youth, he superintended the defenses for three months.*

To follow the fortunes of this distinguished officer leads one from the modest dormitory of the cadet to the palace of the Emperor at the height of his glory. It was Bernard's ability and professional accomplishments which caused his services to be always in demand, yet environment and opportunity played their usual part. Bourrienne, at one time private secretary of Napoleon, in his memoirs, describes how young Bernard first came under the notice of Napoleon:

"At the commencement of the campaign of Austerlitz a circumstance occured from which is to be dated the future of a very meritorious man. While the Emperor was at Strasburg he asked General Marescot, the commander-in-chief of the engineers, whether he could recommend from his corps a brave, prudent and intelligent young officer, capable of being intrusted with an important reconnoitering mission. The officer selected by General Marescot was a captain in the Engineers named Bernard, who had been educated

*The French War Ministry has very kindly furnished for this sketch the following from the official records:

Baron Bernard (Simon). Born the 22d of April, 1779, at Dôle (Jura). Married the 10th of March, 1809, to Marie Anne Josephine Jeanne Népomucène Barbe Crescence de Lerchenfeld. (Marriage license of 7th of February, 1809.)

Sub-Lieutenant student at the Engineer School 21st December, 1796. First Lieutenant 24th December, 1797. Captain 22d March, 1800. Battalion Chief 26th December, 1809. Major 3d August, 1811 (Lieutenant-Colonel). Colonel and Aide to Emperor 21st January, 1813. Field-Marshal 23d July, 1814. In the service of the United States by authority of 2d September, 1816. Returned to active service in the Corps of Engineers 12th February, 1831. Included in the active cadre of the General Staff 22d March, 1831. Lieutenant-General 15th October, 1831. Aide-de-Camp to the King 20th April, 1832. Inspector-General of Engineers, 1834. Member of the Committee of Fortifications 29th October, 1834. Minister of War 10-18 November, 1834. Inspector-General of Engineers 26th June, 1835. Member of the Committee on Fortifications January 1-June 29, 1836. Minister of War 6th September, 1836. Resumed his duties as Aide-de-Camp to the King 31st March, 1839. Member of the Committee on Fortifications 30th April, 1839. Duty at Paris 9th November, 1839.

in the Polytechnic School.* Bernard set off on his mission, advanced almost to Vienna and returned to the headquarters of Ulm. Bonaparte interrogated him himself, and was well satisfied with his replies; but not content with answering verbally the questions put by Napoleon, Captain Bernard had drawn up a report of what he had observed and the different routes which might be taken. Among other things he observed that it would be a great advantage to direct the whole army upon Vienna, without regard to the fortified places; for that, once master of the capital of Austria, the Emperor might dictate laws to all the Austrian monarchy. 'I was present,' said Rapp (then and for a long time previously one of Napoleon's aides), 'at this young officer's interview with the Emperor. After reading the report, would you believe that the Emperor flew into a furious passion? "How!" cried he, "You are very bold, very presumptuous! A young officer to take the liberty of tracing out a plan of campaign for me! Begone, and await my orders.' "

"Rapp told me that as soon as the young officer had left the Emperor all at once changed his tone. 'That,' said he, 'is a very clever young man; he has taken the proper view of things. I shall not expose him to the chances of being shot. Perhaps I shall some time want his services. Tell Berthier to dispatch an order for his departure for Illyria.' This order was dispatched, and Captain Bernard, who, like his comrades, was ardently looking forward to the approaching campaign, regarded as a punishment what was on the Emperor's part a precaution to preserve a young man whose

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**Campaigns.*—Year VII, army of the line, blockade and bombardment of Philipsbourg, affair at the entrenched camp at Manheim; year VIII, reserve army assault of Ivre, where he was the first to enter, attack of the Roman bridge, Battle of Montebello; years IX and X, Army of Italy, passage of the Mincio, siege of Porte-Terrajo; years XII and XIII, army of the ocean coasts (cotes de l'Ocean); Vendemaire year XIV, grand army; 1806 and 1807, army of Dalmatia; engagement of Castel Novo, Italy and Dalmatia; 1809 and 1810, army of Brabant; 1811 to 1814, defense of Antwerp and grand army; 1815, army of the north.

Wounds.—His left arm was pierced by a ball at the engagement of the entrenched camp of Manheim in the year VII; slightly wounded in the passage of the Mincio.

Decorations.—Chevalier of the Legion of Honor, 14th June, 1804; Officer of the Legion of Honor, 24th October, 1813; Commander of the Legion of Honor, 26th May, 1832; Grand Officer of the Legion of Honor, 18th February, 1836; Grand Cross of the Legion of Honor, 9th March, 1839; Chevalier of Saint Louis, 20th August, 1814.

merit he appreciated. At the close of the campaign, when the Emperor promoted those who had distinguished themselves, Bernard, who was thought to be in disgrace, was not included in Berthier's list among the captains of engineers whom he commended to the rank of *Chef de Bataillon*, but Napoleon himself inscribed Bernard's name above all the rest. However, the Emperor forgot him for some time; and it was only an accidental circumstance that brought him to his recollection. I never had any personal acquaintance with Bernard, but I learned from Rapp how he afterward became his colleague as aide-de-camp to the Emperor, a circumstance which I shall now relate, though it refers to a later period.

"Before the Emperor left Paris for the campaign of 1812 he wished to gain precise information respecting Ragusa and Illyria. He sent for Marmont, but was not satisfied with his answers. He then interrogated several other generals, but the result of his inquiries always was: 'This is all very well, but it is not what I want. I do not know Ragusa.' He then sent for General De Jean, who had succeeded Marescot as first inspector of engineers. 'Have you anyone among your officers,' he asked, 'who is well acquainted with Ragusa?' De Jean, after a little reflection, replied, 'Sire, there is a *Chef de Bataillon* who has been a long time forgotten, but who knows Illyria perfectly.' 'What's his name?' 'Bernard.' 'Bernard.' 'Ah, stop—Bernard! I remember that name. Where is he?' 'At Antwerp, sire, employed on the fortifications.' 'Let a dispatch be immediately transmitted, desiring him to mount his horse and come with all speed to Paris.'

"A few days after Captain Bernard was in the Emperor's cabinet in Paris. Napoleon received him very graciously. The first thing he said was, 'Talk to me about Ragusa.' This was a favorite mode of interrogation with him in similar cases, and I have heard him say it was a sure way of drawing out all that a man had observed in any country that he had visited. Be that as it may, he was perfectly satisfied with M. Bernard's information respecting Illyria, and when the *Chef de Bataillon* had finished speaking Napoleon said, 'Colonel Bernard, I am now acquainted with Ragusa.' The Emperor afterwards conversed familiarly with him, entered into details respecting the system of fortifications adopted in Antwerp, referred to the plan of the works, criticized it and showed how he would, if he besieged the town, render the means of defense unavailing. The new colonel explained so well how he

would defend the town against the Emperor's attack that Bonaparte was delighted, and immediately bestowed upon the young officer a mark of distinction, which, so far as I know, he never granted but upon that single occasion. The Emperor was going to preside at the Council of State and desired Colonel Bernard to accompany him, and many times during the sittings he asked him for his opinion upon the points which were under discussion. On leaving the council Napoleon said, 'Bernard, you are in future my aide-de-camp.' "

As shown by the records of the War Ministry, Bernard rose through the various grades to that of field marshal of France in 1814. After Napoleon's retirement to Elba General Bernard gave adherence to Louis XVIII and was appointed a brigadier-general. Upon Napoleon's quitting Elba he again joined his standard and fought with his beloved Emperor at Waterloo. This was to be expected of an old aide-de-camp, and Louis XVIII forgave him, and again permitted him to enter the service of the King, but having received the warning of the Minister of War to depart, he gathered together his collection of engineering plans and data, unequaled in all Europe, and sailed for America.

Under the authority already conferred by Congress, President Monroe, on November 16, 1816, commissioned Bernard to be "an assistant in the Corps of Engineers of the United States, with the rank of brigadier-general by brevet and the compensation that is or may be allowed to the chief of that corps."

The original appointment of General Bernard in the United States Army was specially authorized by Congress and therefore no nomination was sent to the Senate. His name was not borne on the army registers, but in the General Orders of May 17, 1821, his name appears next to that of General Alexander Macomb, Chief of Engineers, as "Assistant Engineer, 16th of November, 1816, Brigadier-General, Brevet."

In a letter dated December 14, 1816, addressed to Major-General Andrew Jackson, at Nashville, Tenn., President James Monroe recited some of the conditions and manner of employment of General Bernard:

> * * * On the subject of fortifications or works of defense of the coasts and frontiers, an arrangement has lately been made by the President, with which I wish you to be well acquainted. You have heretofore, I presume, been apprised that General Bernard, of the French Corps of Engineers, under the recommendation of General Lafayette and many others of great distinction in France

had offered his services to the United States, and that the President had been authorized by a resolution of Congress to accept them, confining his rank to the grade of the chief of our corps. This resolution being communicated to General Bernard by the late Secretary of War, to whom he was known, he came over in compliance with the invitation which accompanied it. From Mr. Gallatin he brought letters stating that he was the seventh in rank in the corps, and inferior to none in reputation and talents, if not the first. It required much delicacy in the arrangement to take advantage of this knowledge and experience in a manner acceptable to himself, without wounding the feelings of the officers of our own corps, who had rendered such useful services, and were entitled to the confidence and protection of their country. The arrangement adopted will, I think, accomplish fully both objects.

The President has instituted a board of officers, to consist of five members, two of high rank in the corps, General Bernard, the engineer at each station (young Gadsden, for example, at New Orleans), and the naval officer commanding there, whose duty it is made to examine the whole coast and report such works as are necessary for its defense to the chief engineer, who shall report the same to the Secretary of War, with his remarks, to be laid before the President. McRee and Totten are spoken of for the two first who, with General Bernard, will continue till the service is performed; the two latter will change with the station. The general commanding each division will be officially apprised of this engagement, and that he may be present when he pleases, and give such aid as he may think fit. The attention of the board will be directed to the inland frontiers likewise. In this way it is thought that the feelings of no one can be hurt. We shall have four of our officers in every consultation against one foreigner, so that if the opinion of the latter becomes of an essential use, it must be by convincing his colleagues when they differ that he has reason on his side. I have seen General Bernard, and find him a modest, unassuming man, who preferred our country, in the present state of France, to any in Europe, in some of which he was offered employment, and in any of which he may probably have found it. He understands that he is never to have command of the corps, but will always rank second in it. * * *

On the day that General Bernard's commission was signed a board of engineers was established, of which he was the senior member during its many years of existence. The duties of this board were to consider all fortifications completed or under construction, then to select sites and make plans for all new works. At its very inception it was provided that: "At those places where naval can come in aid of land defenses the board shall call upon the naval officer who shall have been assigned by the Secretary of the

Navy to co-operate with the board of that station, and who during that co-operation shall be a member of the board."

As if to compensate for former apathy, there was now feverish haste to undertake extensive coast defense fortifications. Within ten days the board proceeded to the northern frontier and began operations at Rouses Point, later going to locate the defenses of the Delaware River, and early in the following year the defenses of Mobile Bay and in the vicinity of New Orleans. General Bernard's next—and among his most valuable—service was as a member of the commission to survey Hampton Roads, York River, and other places in Chesapeake Bay, with a view to selection of a naval depot and to plan a system of defense. The commission recommended two naval depots, one at Burwells Bay on the James and the other at Gosport, with fortifications at Hampton Roads, St. Marys, Elizabeth River and Baltimore. As a general proposition resulting from an extended study the following year (1819) of the coast from Cape Hatteras to the St. Croix River, the commissioners recommended "That Hampton Roads in the south and Boston in the north should be fortified and organized as great naval and military rendezvous, and Narragansett Bay between them as an occasional rendezvous."

His reconnaissances and plans included not only our entire Atlantic and Gulf coast defenses, but the system of national roads, the improvement of interior waterways and a large part of the canal systems which were eventually constructed. Although objected to by his confreres on account of their extent, his plans were used in the construction of Fort Monroe, about the only one of the old casemated fortresses, with moat and drawbridges, posterns and sallyports now garrisoned by United States troops. Few of the multitudes of officers, soldiers and civilians who for nearly a century have threaded their way about the ramparts of this fine example of old French fortification have ever heard of the distinguished exile who so generously gave his talents to the Republic.

The board of which General Bernard was president and the most influential member prepared the project of practically every fortification from Maine to Texas and the surveys and reports on the proposed Dismal Swamp, the Chesapeake and Ohio, the Lake Erie and Ohio, the Alleghany and the Susquehanna, the Susquehanna and Schuylkill, the Delaware and Raritan, the Buzzards and Barnstable Bay and the Narragansett and Boston Harbor canals, and a canal across Florida connecting the Atlantic and Gulf of

Mexico. Their inspections and reports on the improvements of the Ohio, Tennessee, Mississippi and other streams involved an amount of personal work in the span of fifteen years that with the then means of travel would seem an impossible task.

In view of the efforts of the past few years to adjust our army to the changed conditions following the war with Spain it is interesting to read some of General Bernard's reports, particularly one made in December, 1818, on the Military Academy at West Point, in which he stated his views:

"1. That elementary schools are necessary to supply the wants of the army and for the instruction of the militia.

"2. That the elementary schools for the army and those for the use of the militia should be distinct from each other.

"3. That several elementary schools are necessary for the instruction of the militia.

"4. That one elementary school will, in any case at all times, be necessary to supply the candidates for the engineers and artillery, and that in time the same school will be adequate to supply vacancies of the army generally.

"5. That a school of application is necessary for the engineers and artillery departments."

For fifteen years General Bernard had devoted himself to the important duties devolving upon him in the United States Corps of Engineers, when the revolution of 1830 again opened the way for his return to his native land. In a letter dated Washington, July 11, 1831, he informed General Gratiot, Chief of Engineers, that the President "has deigned to accept with a noble and generous kindness" his resignation. It requires little imagination to picture the feelings of this high-minded and talented officer upon finally quitting the friendly shores which had furnished him not only asylum, but a generous opportunity to display his professional ability in the service of a republic which the great Lafayette had deemed it an honor to serve.

The War Ministry records of France had carried General Bernard during sixteen years as absent "In the service of the United States by authority of 2d September, 1816," and upon his withdrawal from the American service he was restored to the Corps of Engineers and included in the General Staff of France. Upon his arrival in France he was promoted to the grade of lieutenant-general and soon after his appointment as aide-de-camp to King Louis Phillipe was announced. General Bernard became inspector-

general of engineers in 1834 and was Minister of War of France from 1836 to 1839. Prior to his death in Paris, November 5, 1839, he was raised to the French peerage with the title of baron. Upon the receipt in the United States of a letter from his son containing the news of his death, the President caused the following order to be issued January 8, 1840:

The President, participating in the sincere grief felt for the death of General Bernard by the officers of the army with whom he was so long associated in the performance of important military duties, and desirous of evincing a proper respect, both for his eminent services to this country and for his virtues as a man, directs that the officers of the army wear the usual military mourning for the space of thirty days from the date of this order.

This, in brief, is the life story of an educated and talented soldier, recognized as the ablest engineer of his generation, who, having served the Emperor until the pall of Waterloo settled over France, declined brilliant offers of employment from European sovereigns and accepted service in the army of the United States. His training and engineering skill were of great moment to the nation when West Point, the Alma Mater of military engineering in America, was yet in its swaddling clothes. His earlier European experiences in campaign and battle were tinged with brilliancy and romance, while his genius laid the foundation of constructive work in America which will live and be builded upon for the benefit of mankind long after the stories of his battles have lost their power to quicken the pulse of a prosaic age.

Failure of Navigable Pass, Dam No. 26, Ohio River*

Dam No. 26, Ohio River, is located about 90 miles south of the town of Parkersburg, W. Va., and about 30 miles north of Huntington in the same State. It is of the usual Ohio River type, adopted for the slackwater improvement of that river. The whole work at Dam No. 26 consists of a lock 600 feet long and 110 feet in width, with straight gates mounted on rollers to allow them to be run in and out of side recesses constructed for that purpose; a navigable pass 600 feet long closed by Chanoine wickets 18 feet in length; two bear-trap dams, each 91 feet in length; and a weir 272 feet long closed by Chanoine wickets 11 feet in length. The three piers for the bear-trap dams have a total width of 38 feet.

While this lock and dam were authorized in 1907, the appropriations were scattered over several successive years, the dam being finally completed in 1911. The work was done by hired labor, directly under the United States Engineer Office with headquarters at Wheeling, W. Va. The foundation of the navigable pass failed on August 8, 1912, and on August 26 the Chief of Engineers appointed a special board of Engineer officers to "report on such matters pertaining to the Ohio River improvement as might be referred to it." The photographs accompanying this report were taken as follows: The first two on August 9, 1912, the day after the failure; and the others on November 9, 1912, after the cofferdam, built to enclose one-half of the navigable pass, had been completely pumped out:

The report of the special board follows herewith in full:

The Board of Officers appointed by Special Orders No. 22, Office of the Chief of Engineers, August 26, 1912, to report on such matters pertaining to the Ohio River improvement as might be referred to it, submits the following report—

The Board assembled at the United States Engineer Office, Wheeling, W. Va., on September 23, 1912, and examined the data in that office relating to the plans, manner of construction, etc., of

*Report of Special Board (appointed by the Chief of Engineers), composed of Col. S. W. Roessler, Col. C. McD. Townsend, and Lieut. Col. Lansing H. Beach, Corps of Engineers.

Dam No. 26, Ohio River, and obtained such information as was possible at that point concerning the failure. The Board, the next day, visited Dam No. 28, where construction was in progress, and examined various features of the work which, it was understood, is in many respects similar to that at No. 26. It being impossible to ascertain the exact conditions at No. 26, the Board adjourned until such date as the cofferdam, then in process of construction, should be completed and unwatered, in order to enable an examination to be made in the dry of the wrecked part of the dam. Owing to several rises in the Ohio River, the unwatering of the dam was not accomplished until November 9, 1912, at which time the Board visited the work and made a careful inspection of so much of the wrecked dam as was included in the cofferdam. The coffer was about 300 feet in length from the lock, and permitted an examination of half of the navigable pass, from station 3+30 to about station 6+30. The Board having considered the information there obtained, subsequently met at the United States Engineer Office, Wheeling, on December 19.

The navigable pass was, in design, the ordinary type of Chanoine wicket dam, which has been used throughout the Ohio River, and is shown in cross-section on Fig. 1 (page 317), the foundation given being that actually constructed at section 5 + 20. The wickets in this dam were 18 feet in length, the crest being at an elevation of 519.5 feet above sea level. The elevation of the lower pool when Dam No. 27 should be completed is estimated at 512.0. The low-water level below Dam No. 26 at present is 505.3. The wickets of the dam were supported on a concrete bed or foundation in form, as shown upon the accompanying illustration (Fig. 2, page 319), which gives characteristic sections of the portion which failed. The average thickness of the concrete varied from 3 to 4 feet, the bottom being at an elevation from 495.8 to 497.2, while the top was at 500.4. The upstream edge had a slightly greater thickness, sloping in a distance of 7.5 feet to an elevation of 502.6. A trench 3 feet wide, with bottom at an elevation of 495.7, was provided near the upper edge as a key where the rock was not excavated to this level. This key near the lock wall had a depth of about 1 foot, but entirely disappeared at station 4+90, and for a larger portion of the navigable pass it had a depth of about 6 inches. The foundation concrete did not abut against the rock immediately above and below it, but was separated from it on each side by an open trench about 1.5 feet wide.

The foundation rested upon soft rock, shaly in character, chemical analysis being as follows:

	Per cent.
Silica	53.10
Oxide of iron and alumina	35.30
Lime	2.60
Magnesia	1.34
Water, loss of ignition and not determined	7.66
Total	100.00

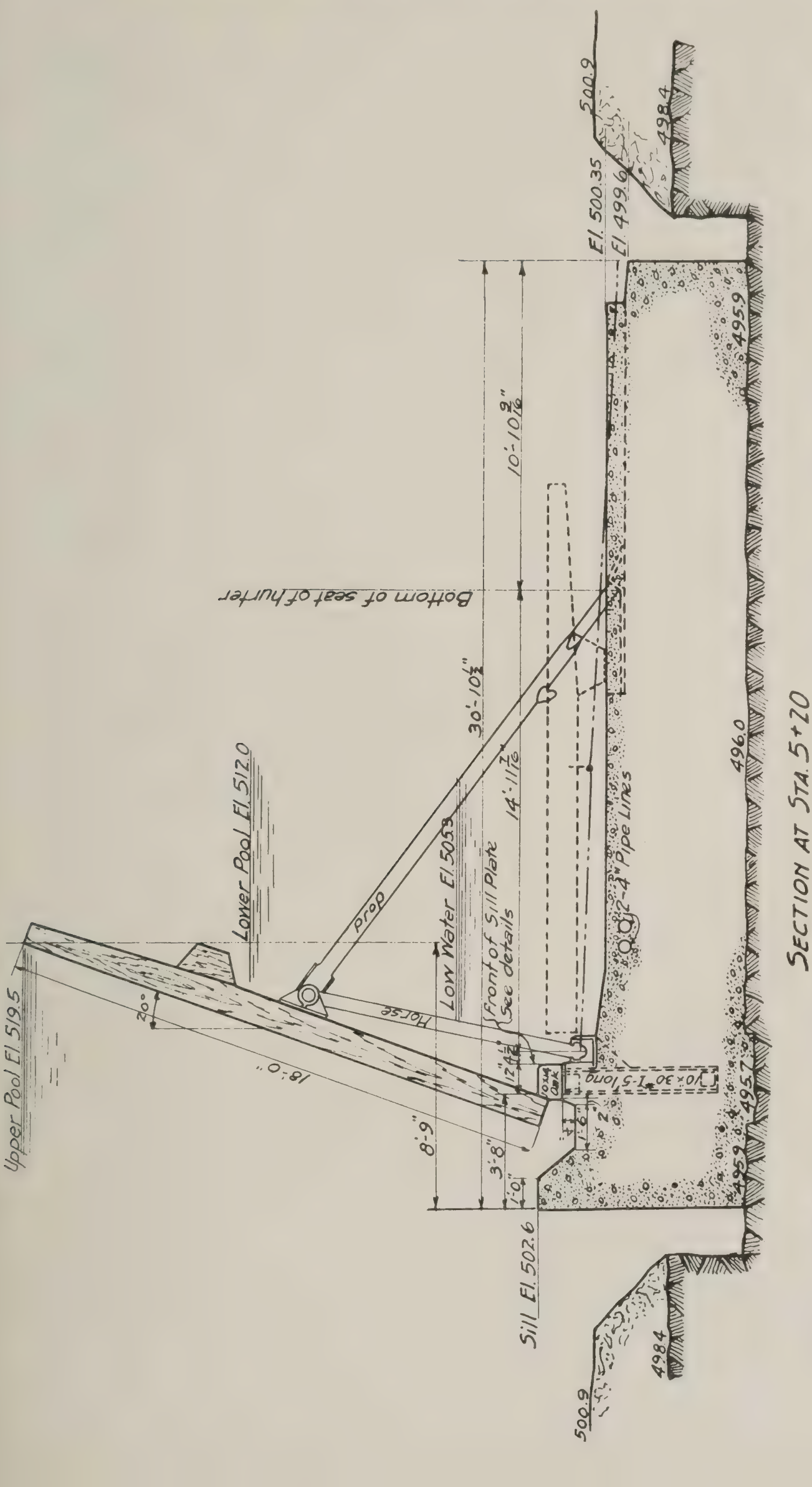

Fig. 1. Cross section through center of wicket at station 5+20, the dividing line between groups F and G, shown in several figures, beginning with Fig. 3.

This rock, of friable character, was removed to a general depth of 2 feet before laying the concrete. When excavated it showed a tendency to split into thin horizontal layers, and this characteristic was readily distinguishable in the bed rock under the destroyed foundation at the time of the Board's examination on November 9. It was smooth and quite greasy to the touch, and evidently has very little frictional resistance and little cohesion. The rock seems to crumble or flake when directly exposed to water and a sample, taken at random by the Board, which appeared to be solid, substantial stone when dry, broke into many small pieces and thin scales after being immersed in water for a few days.

It appears that the dam was first raised on July 16, 1912, at which time the upper pool was filled to within 2 feet of the top of the wickets with a maximum head of about 5.8 feet. It was lowered on July 18, on account of a rise in the river. On August 7, the navigable pass was raised again, beginning at 7.30 a. m., at which time the water was 4.9 feet on the gauge, zero being low water at an elevation of 505.3. The wickets were all raised by noon, at which time the upper pool was 6.5 feet, and lower pool 3.7 feet. One bear trap was raised at 4 p. m. on the same day, and the second one at 8 p. m. The navigable pass failed at 6.30 a. m. the following morning, at which time the gauge readings were: upper pool, 13.4 feet, lower pool 2.7 feet, the head of water being therefore 10.7 feet. The upper pool was at this time 0.8 foot below the top of the wickets. According to eye witnesses, the failure commenced by several wickets in the middle portion of the pass falling as it lowered to the foundation. Immediately thereafter the wickets on each side of these began to move downstream and apparently slightly to the right and left away from a line directly downstream through the point of first failure. The break gradually extended to the pier dividing the navigable pass from the first bear trap and nearly to the lock wall at the other end. One hundred and twenty-nine wickets remained standing in their vertical position, while the remaining 21 disappeared below the surface of the water. The greatest movement of the wickets downstream was 157 feet at a point 455 feet from the river wall, these wickets forming a group of seven, which still remained upright. The position of the parts of the dam after the failure (shown on Fig. 3, page 321) was determined from soundings taken the day of the failure, but the parts within the cofferdam were found to be corrctly located. There was no apparent motion in the foundation under the first ten wickets from the lock wall. The next thirty-two wickets and their foundation slewed downstream around the lock end of the portion as a pivot, the movement of the outer end being between 3 and 4 feet, as judged by the eye. A portion of the dam, thirty-two wickets in length at the midstream end next to the bear-trap pier, swung downstream, apparently without the foundation breaking into separate pieces. The foundation for the wickets, between No. 42 from the lock wall and No. 32 from the first pier, broke

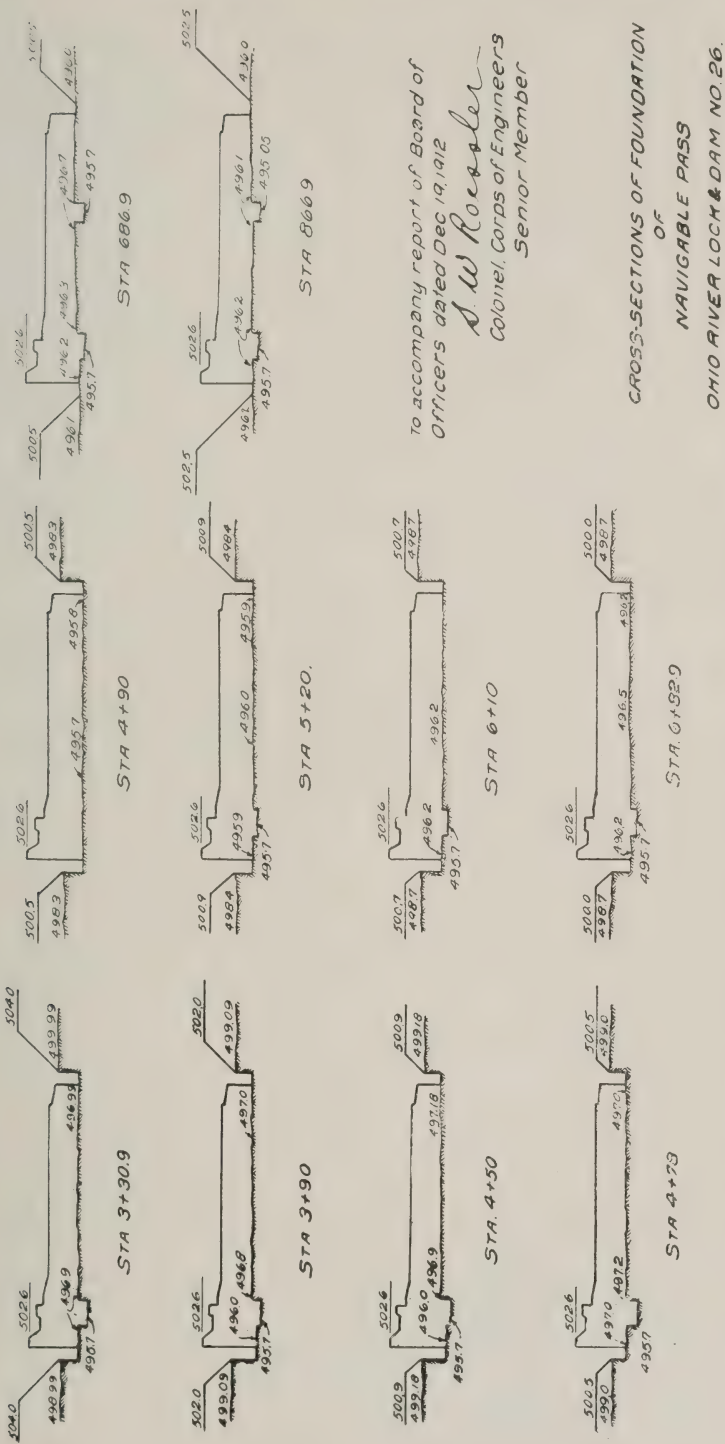

Fig. 2. Stations are numbered from the left, or West Virginia, bank; the pass beginning at station 3+30. (See page 316.)

into six or more pieces, which were moved downstream as above stated.

Some of the detached sections of the concrete foundation were so tilted that the Board had an opportunity of examining the undersurfaces. They were, in some cases, perfectly clean, and in others had a thin layer of rock adhering. This rock in no case appeared to be more than a couple of inches thick. Level lines run over the exposed surface along that portion of the original location of the dam uncovered by the displacement of the foundation, showed a removal of almost uniformly 0.2 foot of rock from what the records state to have been the surface upon which the concrete was laid. On the downstream side of the break there was no abrupt edge of the trench, as indicated in the sections on Fig. 2 (page 319), but the rock sloped up gradually from the original position of the dam as if it had been sheared off by the moving concrete. On the downstream side at the outer end of the thirty-two wicket section mentioned above, where the displacement was small (marked G Fig. 3, page 321), a mass of small, thin, and clayey fragments of rock was found as if heaped up and crushed by the foundation in its movement. The monolith marked F was broken entirely through, the break running along the hurter line. The blocks marked E were also broken in two, the crack following an irregular line. Blocks F had deep, narrow gouges in the upstream edge, termed the guard to the sill, where something must have plowed through with great force, but it is difficult to assign a cause for these cuts. There was considerable abrasion of the upper edge, adjoining the joint between them, produced possibly by the two blocks grinding together. By November 9 nearly all of the wickets left standing after the break, with their horses, had been removed from the water and placed on the river bank, with the exception of the forty-two nearest the lock wall. An examination of those on the bank showed only a single case of severe distortion, and only one case of slight distortion in the metal frames. This absence of damage to the wickets or horses would indicate that the disrupting force did not act upon them but exerted its effect at a lower level. The quality of the concrete of the foundation in all cases appeared good, and the separations appear to have occurred mainly at the joints left between successive construction sections or monoliths.

It was at first thought that some information concerning the method of breaking and the cause of failure could be learned from the fact that the first movement noticed was the falling of several wickets in the middle of the pass, but investigation develops the information that wickets not infrequently fall without any apparent cause, as the head increases while the pool is being formed. The falling in this case, therefore, while evidently marking the beginning of the movement, can not be considered as conclusive evidence of the action occurring or even of the motion involved.

The effect of conditions, such as prevailed in this locality, can

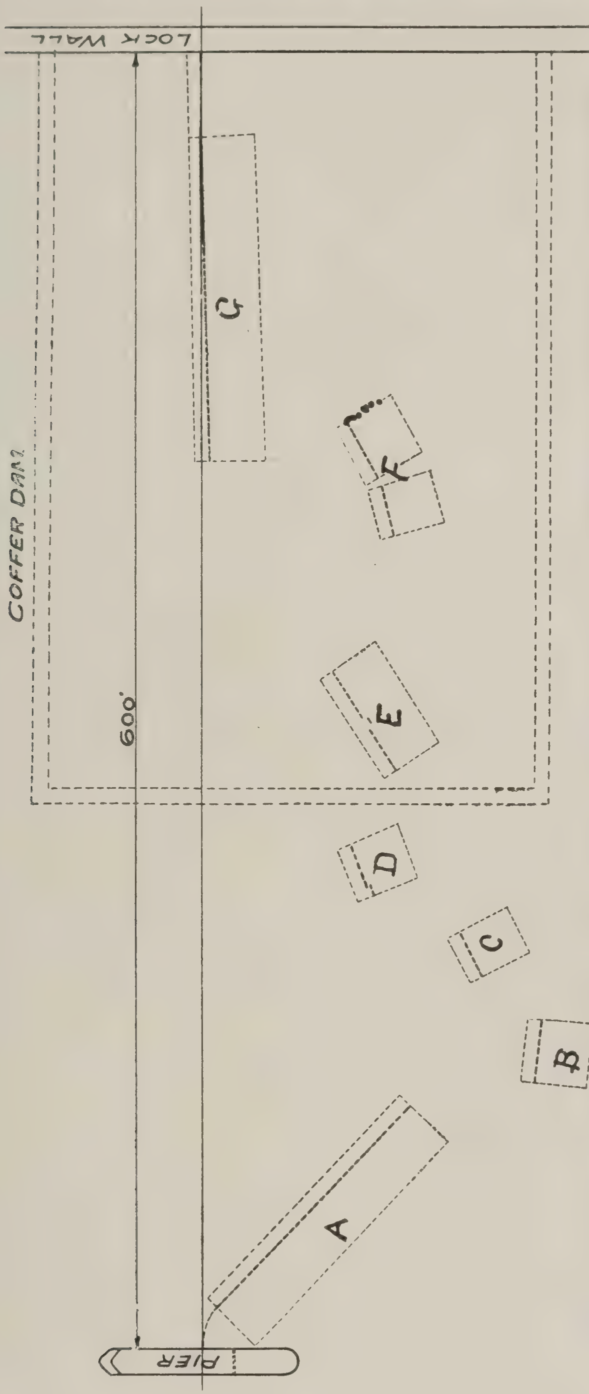

Fig. 3. Ohio River Lock and Dam No. 26. Position of foundation of the navigable pass after failure, August 8, 1912. (See page 318.)

not be more definitely or clearly stated than has been done by Trautwine in his Engineer's pocket-book when he says:

"The assumption of ideal conditions is particularly dangerous in the case of masonry dams. Thus, any compression of the material at the downstream face may open seams on the upstream face; and water, entering these seams, will exert a wedge-like action, shifting the resistance line further downstream, thus still further increasing the tendency to crushing on the downstream face and to opening on the upstream face. Again, if any relatively smooth joints have been left, the water, thus penetrating into or under the dam, increases the tendency to slide, not only by diminishing the effective weight of the upper portions, but also by acting as a lubricant upon the seam where it penetrates.

"A retaining wall may slide, without losing its verticality; and, indeed, without any danger of being overturned. This is very apt to occur if it is built upon a horizontal wooden platform; or upon a level surface of rock, or clay, without other means than mere friction to prevent sliding, * * * and on *wet clay*, friction may be as low as from 0.2 to 0.333 the weight of the wall."

Either assumption would account for the accident. The rock upon which the concrete rested had a very small frictional coefficient, and was composed of layers through which water could penetrate and produce an upward pressure on the base of the dam.

The weight of practical experience in dam building confirms the view that the reliability of rock as foundations for dams is easily or frequently overestimated after even a careful examination of its physical properties and a chemical analysis of its contents, and the Board is therefore of the opinion that to guard against a repetition of the failure, in future construction, a greater thickness should be given to the concrete foundation, so as to insure sufficient weight to counteract any possible water pressure under it; that greater care should be taken to bond the concrete to the rock by a key having a depth of at least 5 feet, where the thickness of concrete is not for other reasons increased, and by the concrete entirely filling the foundation trench; and that, as an additional precaution, the concrete be also securely fastened to the rock by anchor rods. The use of channelling machines to form the sides of the trenches is also recommended to prevent any possible opening of seams in the rock by blasting.

(Signed)

S. W. Roessler,
Colonel, Corps of Engineers.
C. McD. Townsend,
Colonel, Corps of Engineers.
Lansing H. Beach,
Lieut. Col., Corps of Engineers.

COMMENTS

Maj. F. W. ALTSTAETTER*
Corps of Engineers

A few remarks in addition to the report of the Board and the introduction thereto might be of interest.

The pass was designed in 1908, and the drawings showed a "minimum" section of concrete. They showed a key of concrete

Fig. 4. View of Dam No. 26 the morning after failure. Taken from left bank of the river in West Virginia; the camera was on the center line of wickets, the line being shown broken in illustration. Some of the wickets in group G, next to the lock wall, had been lowered before the picture was taken. The groups are lettered alike in all illustrations.

with the bottom at elevation 495.7, as shown on Fig. 1. The key was 3 feet wide. Except for the key, the bottom of the concrete was shown horizontal at elevation 498.3, so that the key was 2.6 feet deep and most of the main slab of concrete was only 2 feet thick. The following note was added under the concrete: "Loose

*In charge of the Wheeling District.

and soft shale to be removed. Foundation to be on solid rock. Foundation bed to be rough and concrete to be bonded to rock.'' The exact elevation of the rock was unknown, and it was intended to make the concrete as much thicker than the minimum section as conditions required.

The calculations for stability were made for the minimum section. The upper pool was assumed full and the lower pool at extreme low water. Such severe conditions will probably never occur. When the river reaches extreme low water discharge, it is difficult to keep the upper pool full and, unless Dam No. 27 were in operation, there would be no special reason to do so. However, No. 27 would probably be up, providing 6.7 feet more water than low water at No. 26: and even No. 28, 33 miles away, would raise the water 0.3 foot above low water if its full pool were perfectly horizontal and No. 27 were down. Then, again, extreme low water seldom occurs, perhaps once in twenty years. No other possible conditions are as severe as those assumed.

In the calculations, water pressure upward on the concrete foundation was assumed to vary from full upper pool at the upstream edge to low water at the downstream edge of the base. The bond between concrete and rock was assumed such that only 50 per cent of this pressure would act.

The weight of the horses, props, and imbedded steel was neglected. The resultant pressure pierced the base 1.3 feet inside the middle third.

The 50 per cent bond seemed rational. It was known that considerable rock adhered to the concrete when the gate track was removed from similar rock at Dam No. 37, Ohio River. It is believed this also happened generally at No. 26. Where the concrete foundation was discovered free from rock, it had moved downstream considerably, which may have rubbed the rock off. Either slab ''E'' or ''F,'' Fig. 3, was blown up with dynamite and rock was found adhering to the fragments. A survey of the pass also indicated movement of the foundation rock with the concrete. Soundings were taken by the lockmaster with a pole and readings checked by Capt. M. C. Tyler, Corps of Engineers, who was in the boat. The readings checked up absolutely when they fell upon the pass sill or other points whose elevation was known. The lowest rock excavation actually made was 495.0 feet, and was in the key at section 8+6.9 (Station 3+30.9 was next to the river wall and 9+30.9 next to the bear-trap piers). The elevations of the rock surface were carefully taken just before the concrete was placed. The surveys after the failure show elevations of rock or gravel which some places overlay the rock. Between Stations 7+46.9 and 8+66.9 no rock was discovered above elevation 495.4 and some were as low as 493.8. This would indicate that about 1 to 2 feet of rock went out with the concrete at the point where the pass moved farthest downstream and the point of the initial failure. There were three or four points where the elevation of the rock was

Fig. 5. View taken soon after Fig. 4, but from opposite side of the river. The power-house and keeper's quarters are shown on the left bank of the river. The foot-bridge in foreground is on "A" frames and is for the purpose of raising and lowering the weir wickets, the tops of some of which can be seen between the "A" frame foot-bridge and the derrick boat. The three abutments are for the bear-trap dams.

taken inside of the present cofferdam when the Board was present and these elevations gave the results noted in the Board's report.

As to the quality of the rock, it impressed those of us on the job during construction much more favorably than it did the Board after failure. This is natural. Moreover, at the time of the Board's inspection the river was rising rapidly and it was necessary to flood the cofferdam before the rock had been properly cleaned of muck, etc., except in a few spots. So the Board did not have as good an opportunity for examining the rock in place as I should have liked. Except for a little sandstone near the lock, it was all a superior blue shale, perfectly homogeneous, without any flaws or breaks. The top was loose and scaly and some of it disintegrated badly when subjected to water, but no rock that was known or suspected of being such was allowed to remain in place. It was removed until it came out in irregular pieces and until it was believed all poor or seriously stratified rock had been removed. Quite a large number of engineers visited the work, some of them very good ones (it is unnecessary to mention names), but I have no recollection of any of them questioning the quality of the foundation. A sample was blasted at random from the foundation, after the failure, and sent to the United States Geological Survey for examination. The following letter was received in reply.

Department of the Interior,
United States Geological Survey,
Washington, January 11, 1913.

Lieut. Col. H. C. Newcomer,
Corps of Engineers,
Washington, D. C.

Sir: In reply to your letter of December 26, asking for a mineralogical analysis of a sample of rock from the foundation of Dam No. 26, Ohio River, about 9 miles below Gallipolis, Ohio:

I take pleasure in sending you the following notes based mainly on a microscopic examination by Mr. E. S. Larsen, Associate Geologist:

The rock, of which a specimen was submitted, is shale and about half of it consists of soft mineral kaolinite, the other minerals present being quartz, muscovite and chlorite. The quartz is in grains, for the most part less than one-tenth of a millimeter in diameter, and the muscovite (white mica) is in minute shreds showing but little parallel orientation. The rock probably belongs to the Conemaugh formation of Pennsylvanian (Carboniferous) age.

Shales are in general among the weakest rocks, especially when saturated with water. However, the present shale, in spite of the large proportion of kaolinite, is not particularly fissile, and appears to be somewhat stronger rock than the average shale. It is not quite clear from your letter whether this rock represents material used in the construction of the foundation of the dam, or

whether it is the bed-rock in place under the structure. If it is the rock in place, and the dam was built directly upon it, petrographic examination suggests no reason why the material should fail, unless the rock under the dam is traversed by faults so that portions of it are crushed or would admit the water through open crevices. In that case, large proportions of kaolinite in the rock

Fig. 6 (upper). View from river wall of lock looking a little downstream and showing coffer around one half of pass after being unwatered November 9, 1912. The foundations of groups E, F, and G, Fig. 3, can be seen. The mud is nearly all cleaned off the wickets of Group G. These wickets were all lowered soon after the failure. Wickets of groups E and F had been removed before the coffer was unwatered.

Fig. 7 (lower). View taken from near the center of top of downstream coffer, showing groups E and F and a portion of G.

and its tendency to split into flakes would constitute important features of structural weakness.

Very respectfully,

GEO. OTIS SMITH,
Director.

No fault in the rock ever came to my attention, and I visited the work about twice a month during construction and generally stayed several days.

Before placing concrete, this rock was freed of all loose pieces, thoroughly cleaned, and the surface left rough and irregular. Both Captain Tyler and Mr. Cutting (one of whom was in local charge at all times) were particular about the rock being clean and rough. I discussed the matter with both of them repeatedly.

Fig. 2 might give the impression that the trench was designed absurdly shallow but, as explained earlier, the elevation of the bottom of the trench was fixed and the rest of the rock excavated as necessary. In several sections a small second trench is shown which was introduced by Captain Tyler and Mr. Cutting, to roughen the rock surface.

The total horizontal downstream pressure on the pass under the most unfavorable circumstances is only about six short tons per linear foot, caused by a head of 14.2 feet—surely not a big pressure. And at the time of failure it was much less. In my opinion, the failure is clearly one of the foundation rock shearing and sliding on itself. The mistake was made in trusting the rock, although we thought we were not trusting it to withstand very much. Water probably percolated between the layers of the rock, as the failure is hard to explain otherwise. This, in spite of the fact that our observation during the construction led us to a contrary view.

The original cost of the entire navigable pass, exclusive of the movable parts, was $61,091.00. The movable parts have nearly all been recovered and the cost of some of the rock and other excavation is not lost. Some horse boxes, sill plates, sill timbers, etc., will be salvaged. When the new pass which will replace this one is built, the final cost will probably be about $50,000.00 more than if it had been so built originally.

It may be interesting to state that the new design contemplates a key 6 feet wide and 5 deep, which dimensions are to be kept constant regardless of the thickness of the slab. The concrete is to be 6 feet longer up and down stream, and the key and the rest of the slab of concrete are both to be anchored to the rock by steel rods. These rods will be numerous and will also serve to bind the upper layers of rock together. (See Fig. 4, page 149, No. 20 PROFESSIONAL MEMOIRS for March-April, 1913.—ED.)

Fig. 8 (upper). Drainage ditch, about 50 feet downstream from the foundation of the pass, used in unwatering the coffer. Rock apparently exactly same as under foundation.

Fig. 9 (lower). A portion of foundation on which group E originally rested. Note material appears of same shaly nature as seen in Fig. 8.

The New Quadrant Protractor

BY

Maj. R. R. RAYMOND
Corps of Engineers

Hydrographic surveyors are familiar with the three-point problem, but for the sake of the reader who has not had occasion to use it in recent years it may be well to state it here. If three points A, B, and C have been established and have been plotted on a map, and the angles ASB and BSC be measured at a point S, it is required to plot S on the map.

This problem frequently arises when soundings are taken from a boat and the angles to the stations A, B, and C on shore are measured by means of a sextant in the boat.

There are several well-known solutions of the problem, of which the most commonly used is the mechanical solution with the three-arm protractor. In this solution, the two movable arms of the instrument are set at the proper angles and the protractor is then moved about over the map until the straight edges of the three arms pass through the plotted positions of the three known points, under which condition the center of the protractor indicates the position of the desired point. To facilitate the moving about of the instrument, a pin is set in each of the plotted known points, two of the arms are brought into gentle contact with the proper two pins and the protractor is carefully swung around until the third arm touches its respective pin. The center is then marked.

This solution has generally been considered the best and most convenient available. It has, however, some disadvantages. For instance, the instrument is quite expensive, costing ordinarily $75.00 or more. It is large and heavy, consists of a number of losable parts and requires care to keep it in order. Since only those parts of the arms which are outside of the graduated circle can be used ordinarily, the center can approach any station pin only to a distance greater than the radius of the circle. This necessitates the use of a rather small circle and the graduations

are consequently so fine that reading glasses are necessary for setting the verniers.

On the other hand, the distance from any station to a point whose location is to be plotted is limited to the length of the arm. Extensions giving a reach to near 3 feet are often used. This makes the instrument awkward to handle, and a large drafting table entirely clear of the usual small articles and implements, even of the thumb tacks to hold the map, is required.

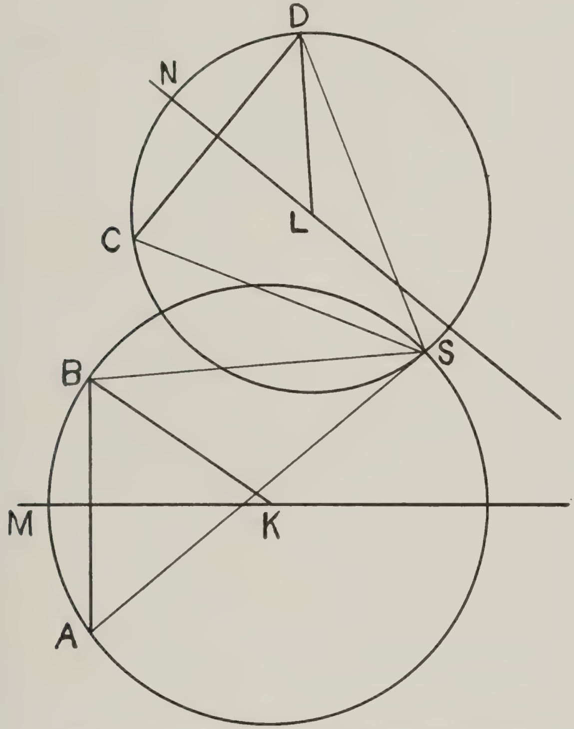

Fig. 1. General solution of 4-point problem.

The pressure of the arms against the station pins causes the arms to spring slightly with an appreciable effect on the accuracy of the plotting; but this is not a serious matter in ordinary hydrographic work where such a small error in the location of a sounding is not important.

In spite of the above shortcomings, the three-arm protractor has had a long and successful career.

It happened one time that through an error on the part of an observer the proper angles for the three-point problem were not

measured, and a *four-point* problem resulted. Thus, one angle ASB (Fig. 1, page 331) was properly measured, but the second angle actually measured was CSD. The error was repeated for a large number of soundings.

The writer adopted the following solution: Draw chords AB and CD and erect a perpendicular at the middle point of each. Through B draw BK, making the angle BKM equal to the given angle ASB. With K as center and KB as radius, draw a circle passing through A and B. The circumference is a locus of S, for if we assume any point S′ on the circumference and draw lines S′B and S′A, we shall have the angle AS′B measured by one-half the arc AB, and as the angle BKM is measured by one-half the same arc, the angle AS′B must be equal to angle BKM and, consequently, to the given angle ASB. It can be shown readily that no point not on the circumference can fulfill this condition. Thus, the circumference is the locus of all points where the angle ASB is equal to the given angle.

In the same way draw DL, making the angle DLN equal to the given angle CSD. The circumference whose center is L and radius LD passes through C and is the locus of all points where the angle CSD is equal to the respective given angle.

The intersection of the two circumferences locates the point S. Since circumferences intersect in *two* points, there are two possible locations of the point S which fulfill the conditions. This is true whatever method of plotting is used, but the correct point may be selected generally by inspection.

It is evident that the three-arm protractor can not solve directly this four-point problem. When the problem arose in practice, it was solved as stated, the operations being expedited by means of a device which it is unnecessary to describe here, but to reduce the problem to its simplest terms a special protractor was designed later which is the subject of this article.

Fig. 2 (page 333) shows this protractor as made by Alteneder & Sons Co. of Philadelphia. It differs from the author's original design in the position of the arc only. The essential feature of the instrument is that it is graduated to read the *complement* of the angle between its arms.

Referring to Fig. 1, it is evident that if this protractor be centered on B and one arm be placed on BA and the other arm on BK, then the vernier will read directly the complement of the angle ABK, which is the angle BKM. In other words, the arm bearing the arc intersects the perpendicular to the chord at the

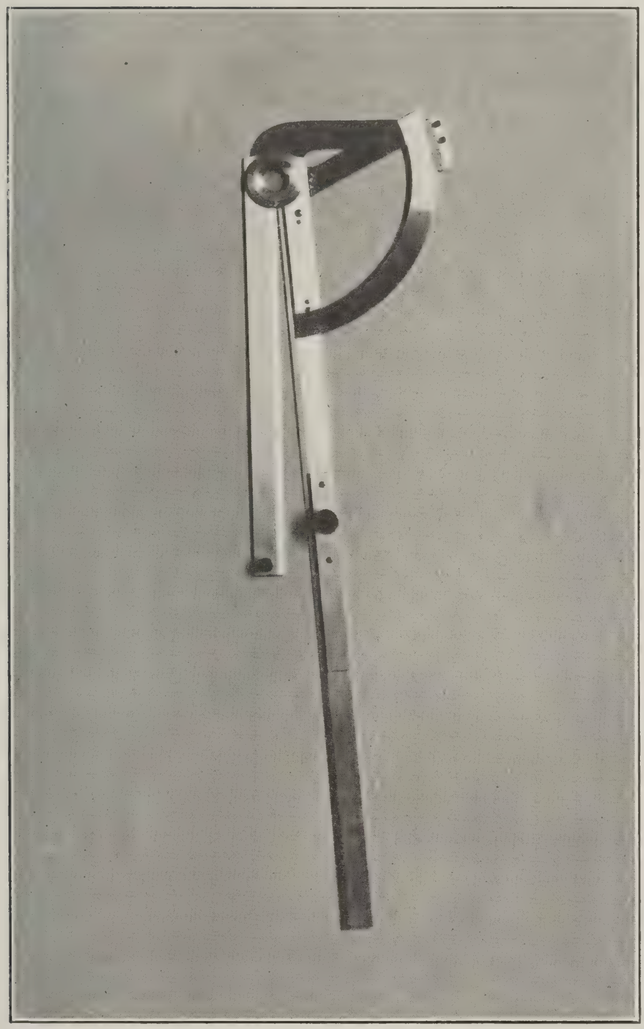

Fig. 2. Quadrant protractor.

center of the circle which is the locus of all points S where the angle ASB is equal to the vernier reading.

With this protractor the center of the desired circle is found without the use of any construction lines except the perpendicular at the middle point of the chord, and this is drawn once only for all points whose positions are referred to the stations A and B.

Points are completely plotted by drawing the arcs whose intersections are desired. Only very short arcs need be actually drawn.

If any two of the four stations coincide, the solution of the problem with this protractor is unchanged. This gives the three-point problem as described above.

If only two station points are used, the locus of the point S can still be determined, but a complete location of the point S can be made only by constructing another locus to intersect the first. This second locus may be an arc or it may be a right line, such as a range or a transit sight.

While several different problems arising in hydrographic surveying can be solved by this new protractor, its function in all cases is the same: to determine directly the center of the circumference which is the locus of all points S at which the angle ASB between two given points A and B is given.

A knowledge of the capabilities and limitations of the instrument is best obtained by comparing it with the three-arm protractor, for which purpose a series of tests was made. The three-arm protractor used was a new one by a well-known maker and seemed to be an excellent instrument. It was purchased for regular use in a United States Engineer office.

Comparing the instruments by inspection, we note the following:

	Three-arm Protractor.	New Protractor.
Weight of protractors as shown in Fig. 3	62.5 ounces	13.75 ounces
Length of arm with extension	32.5 inches	18.75 inches
Reach	3.65 to 32.5 inches	0 to 37.5 inches
Cost	$75.00	$25.00 (estimated)

Comparative tests were made on a large sheet of paper, so that the points to be plotted would lie over the entire fields of both instruments. Fig. 3 (page 335) shows the two protractors on the same scale. It was concluded from the tests that the new protractor would be more generally useful if made with an extension arm 22 inches long. This would be only 3.25 inches longer than the one tested.

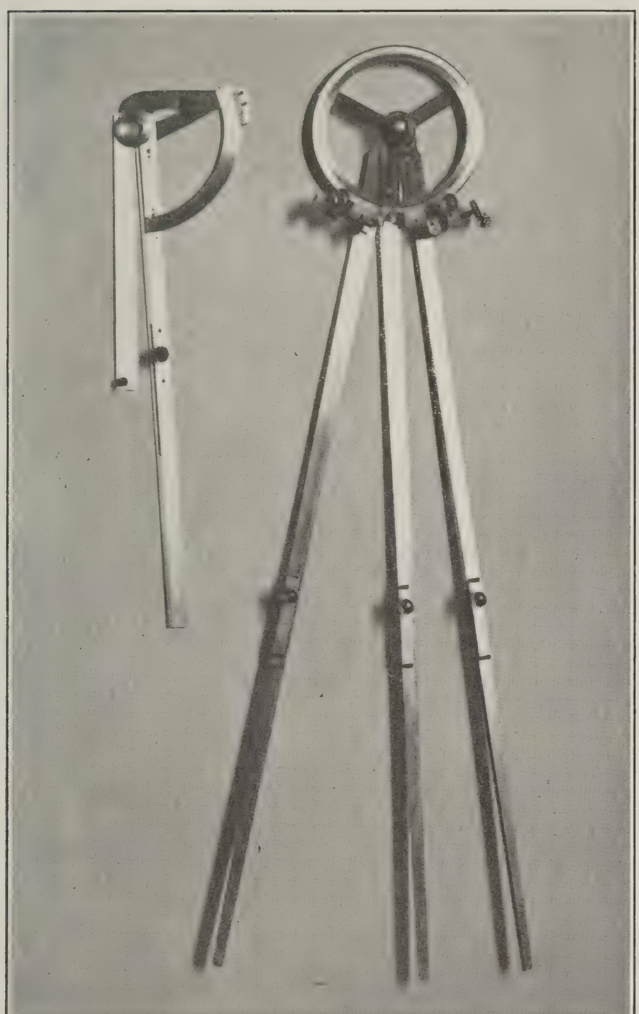

Fig. 3. Comparative sizes of quadrant protractor and 3-arm protractor to cover practically same area of map.

In Fig. 4 (page 337) are shown the fields of the three-arm protractor and the 22-inch new protractor. These fields vary considerably for different relative positions of the assumed station points. In this case the station points were taken 10 inches apart on a circle of 22 inches radius, which gives the maximum field for the 22-inch new protractor. This protractor does not give good results in plotting points near this limiting circle, for the intersecting arcs used in plotting are too nearly concentric to give good intersections. The zone of inaccurate work is, however, quite narrow; reasonably accurate results having been obtained in the intersection of two arcs of 22 inches radius whose centers were only 1 inch apart. Furthermore, it must be remembered that by piecing out the arm of the protractor by a straight-edge its field may readily be greatly enlarged.

The full, heavy line shows the field of the three-arm protractor, while the broken line shows that of the new protractor.

Two points having been assumed as stations, points lying on a right line crossing the field from within the range of both instruments to beyond their extreme reach were plotted in turn by both protractors. The new protractor plotted points outside of the field of the other. Among twelve points plotted, four intersections fell so exactly on the correct points that the errors could not be determined, while the maximum error was 0.07 inch and the average for all was 0.015 inch.

The three-arm protractor plotted ten of the same points, its least error being 0.02 inch, maximum 0.28 inch, and average 0.074 inch. The springing of the three arms when pressed against the station pins is believed to account for the inaccuracy of the results.

In point of speed, the two instruments were very much alike, the average time per point differing by only about 5 per cent in favor of the new protractor. Other tests gave the greater speed to the three-arm protractor, but in no test did the speeds differ by more than 5 per cent.

This result is somewhat surprising in view of the fact that the new protractor requires both the setting of an angle and the striking of an arc; but the causes are readily found. The arc of the new protractor is larger than that of the other; the graduations are coarser, although having the same least count; and so the angles are set without the use of a reading glass. In addition to this saving in time, the three-arm protractor requires careful handling to bring it into the exact position to plot each and every

point, while the new protractor is set for one pair of stations, such as A and B, and the angles for a number of points are set in rapid succession without disturbing the instrument, each center for an arc being given a mark for identification. The arcs are then struck and the proper intersections marked.

Thus, while the three-arm protractor can plot a single point more rapidly than the new protractor, its advantage in speed is lost when about five or more points are to be plotted.

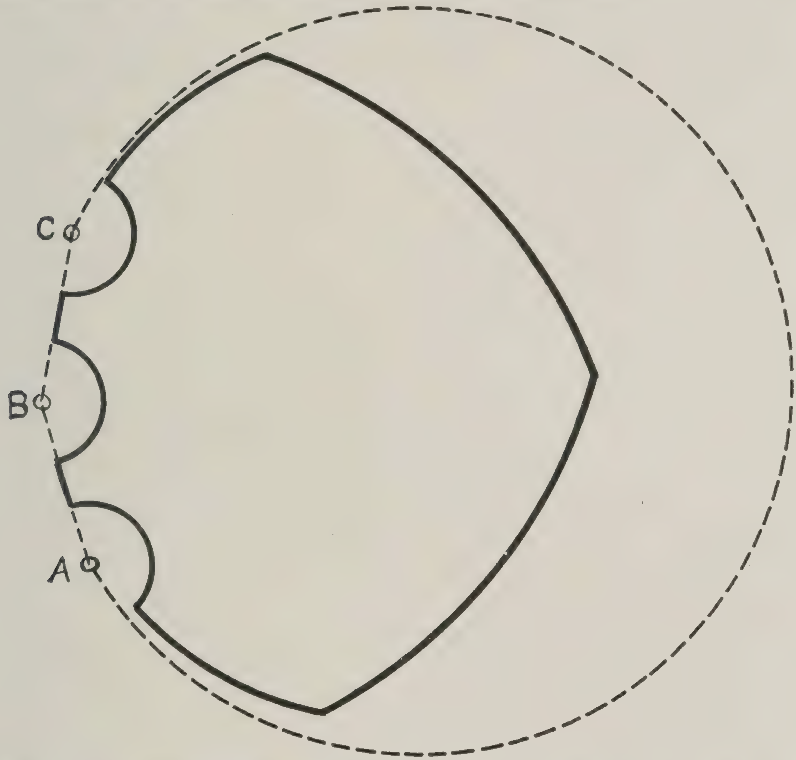

Fig. 4. Comparative areas that can be plotted on a map with the 3-arm protractor (solid line) and the quadrant protractor (broken line) shown in fig. 3.

The two protractors were compared further in plotting a number of points by the three-point problem. In this case, the speeds were again nearly equal, the advantage being with the three-arm protractor. In all the tests no effort was made to obtain maximum speed, but the work was done uniformly and as nearly like regular work under service conditions as possible.

Of eleven points plotted by the new protractor, one was as exact as a pin-point could be placed on the map; the maximum error was

0.22 inch and the average was 0.044 inch. The least error of the three-arm protractor was 0.02 inch, the maximum 0.31 inch and the average 0.105 inch.

In this test the new protractor made much the better showing. Omitting the point where the error was 0.22 inch, this being the most distant point reached and the errors of both instruments being the same, the ten remaining points were plotted with a least error of 0.0 inch, a maximum error of 0.9 inch and an average error of 0.027 inch. The three-arm protractor plotted nine of the same points with a least error of 0.02 inch, a maximum error of 0.31 inch and an average error of 0.092 inch. It failed to plot the tenth point.

A number of other comparative tests were made and in each case it was found that the speeds were about equal and either protractor was accurate enough for the class of work done. The new protractor had a decided advantage in area of field and in convenience. When not needed on the drawing table, it was readily pushed aside or placed on another table or shelf or hung on a nail. The large and delicate three-arm protractor required more or less elaborate preparations for its reception and it had to be handled with care to avoid a disaster.

Returning to Fig. 1, it should be observed that points S lying within a circle having AB for its diameter will have the angle ASB greater than 90 degrees. In this case, the supplement of the angle is taken and plotted by the protractor turned around so as to mark the desired center on the perpendicular on the other side of the chord AB.

Plotting many points involves replacing the protractor frequently upon the stations and chords. To facilitate this, a notch is provided through which a pin may be set into the drawing board after the instrument is correctly centered and aligned. A second pin is driven into the board against the back of the fixed arm near its outer end. The protractor may then be freely removed and replaced at will without loss of time or accuracy of centering and alignment.

Great Falls Power

BY

Lieut. Col. W. C. LANGFITT
Corps of Engineers

The act of Congress providing for the expenses of the District of Columbia for the fiscal year ending June 30, 1913, contained an item providing for the investigation of the practicability and advisability of developing water power at or near the Great Falls of the Potomac, to supply the needs of the General and Municipal Governments, combined with an increase in the water supply of the City of Washington. These investigations were assigned to the writer. In the investigation and preparation of the report, he was assisted by Mr. Clemens Herschel, Mr. Walter C. Allen, of the District Government, Captain Jarvis J. Bain, Corps of Engineers, and Mr. E. D. Hardy, Superintendent of the Filtration Plant. Mr. Waddy B. Wood prepared the plans and estimates for the substations, and much valuable information was obtained from the General Electric Company as to the electrical installation.

The following article is a brief and somewhat hurried abstract of the full report. Sixty-four drawings accompanied the report and a full set of general specifications—only a few of the drawings can be shown and the specifications are necessarily omitted. Both are of course subject to minor change when actual work is proposed.

WATER SUPPLY.

The first investigation for an increased water supply to the City of Washington was carried out by Maj. Jay J. Morrow, Corps of Engineers, under authority of an act of Congress approved May 26, 1908. The report is printed in House Document No. 347, 61st Congress, 2d Session. The only project favorably considered in this report was that of building a new conduit and tunnel from Great Falls to McMillan Park Reservoir (parallel line project), but as, in the meanwhile, the Patuxent River had been suggested as a source of supply, no definite recommendation was made pend-

ing an investigation of that river. The latter investigation was made by the writer under authorization in the District of Columbia Bill approved March 2, 1911, and report thereon is printed in House Document 1266, 62d Congress, 3d Session. This report shows that an average of 90,000,000 gallons of water daily could be depended upon in the execution of the plans suggested and that the cost was reasonable. No definite recommendation was made as, in the meantime, the possibility of combined power development with increased water supply from the Great Falls of the Potomac had arisen. The report stated in substance that, could a power project and increase of water supply be combined economically, the Patuxent River as a source of supply would necessarily be eliminated. The studies of this combined project are the ones herein considered. Fig. 1 (page 341) shows the general layout of all these projects, as well as that of present water supply.

POPULATION AND CONSUMPTION.

The following table, showing the population of the past years and the estimated population up to 1950, together with the actual consumption to 1912 and the estimated consumption up to 1950, was compiled. From it, it was decided to estimate the average per capita consumption in the future to be 173 gallons daily, and that the maximum per capita consumption was estimated to exceed the average by 30 per cent. Table 1:

Fiscal year ended June 30	Population.	Daily consumption—Average.	Daily consumption—Maximum.	Average gallons per capita daily.	Percentage of increase of maximum over average.
		Gallons	*Gallons*		
1905	323,000	68,700,000			
1906	326,000	67,400,000			
1907	330,000	66,900,000	80,290,000	203	20
1908	339,000	64,910,000	80,380,000	192	24
1909	343,000	61,470,000	78,930,000	179	28
1910	343,000*	59,190,000	78,500,000	173	33
1911	348,000	60,380,000	78,320,000	173	29
1912	354,000	62,120,000	92,720,000	176	49
1915	378,000	66,000,000*	86,000,000*	173‡	‡30
1920	405,000	70,000,000†	91,000,000†	173‡	‡30
1930	460,000	79,000,000†	103,000,000†	173‡	‡30
1940	515,000	89,000,000†	116,000,000†	173‡	‡30
1950	570,000	99,000,000†	129,000,000†	173‡	‡30

*Since United States census is consistently lower than the police census, the per capita consumption is based on the latter for 1909, viz, 343,000.

†Estimated from probable future population, assuming a per capita consumption of 173 gallons per day and a maximum consumption 30 per cent greater than the average.

‡Assumed.

Fig. 1. General map.

SUPPLY AVAILABLE IN POTOMAC RIVER.

The discharge of the Potomac River from fifteen years' observations by the United States Geological Survey varies between 600,000,000 and 84,000,000,000 gallons daily. As the predicted maximum daily consumption in 1950 is only 129,000,000 gallons, it may be stated that the Potomac River at Great Falls can furnish for all time a sufficient water supply to the city and that until long after the usual limits of predictions have been passed a surplus will be available, even in times of excessive drought.

CAPACITY OF PRESENT SUPPLY.

The maximum delivery at the present time is 90,000,000 gallons daily, and this can be allowed but for a very short time on days of maximum consumption—this mainly on account of reduction of storage necessary to attain it, and because the pumps supplying the filters were designed to handle safely only about 75,000,000 gallons daily. While the pumps can be pushed to 90,000,000 gallons delivery, such a course involves risk not lightly to be undertaken under present conditions. The filter beds aggregate 29 acres in extent, and it has been found that with the addition of the coagulating plant now in use, and on account of their inherent capacity, they can be safely run at a rate of about 150,000,000 gallons per day. The present filtered water reservoir, with a capacity of 15,000,000 gallons, will be too small with increased consumption, as will also the present piping, designed for 75,000,000 gallons daily.

PLANS CONSIDERED.

Of the plans for increased water supply previously reported upon, but two are worthy of consideration—these are the parallel line project of Major Morrow and the Patuxent River. These are designated as project No. 1 and project No. 2, and the method of water supply combined with power development, the subject of this article, is project No. 3.

The plan recommended for power development and increasing the water supply includes the construction of a high dam located in the Potomac River almost on the northwest boundary line of the District. This dam will form a large lake, and from this, by means of hydraulic machinery, an increased water supply is to be pumped into the Dalecarlia Reservoir. The adoption of this method will involve enlarging the capacity of the delivery between the Dalecarlia Reservoir and the Georgetown Reservoir and, similarly,

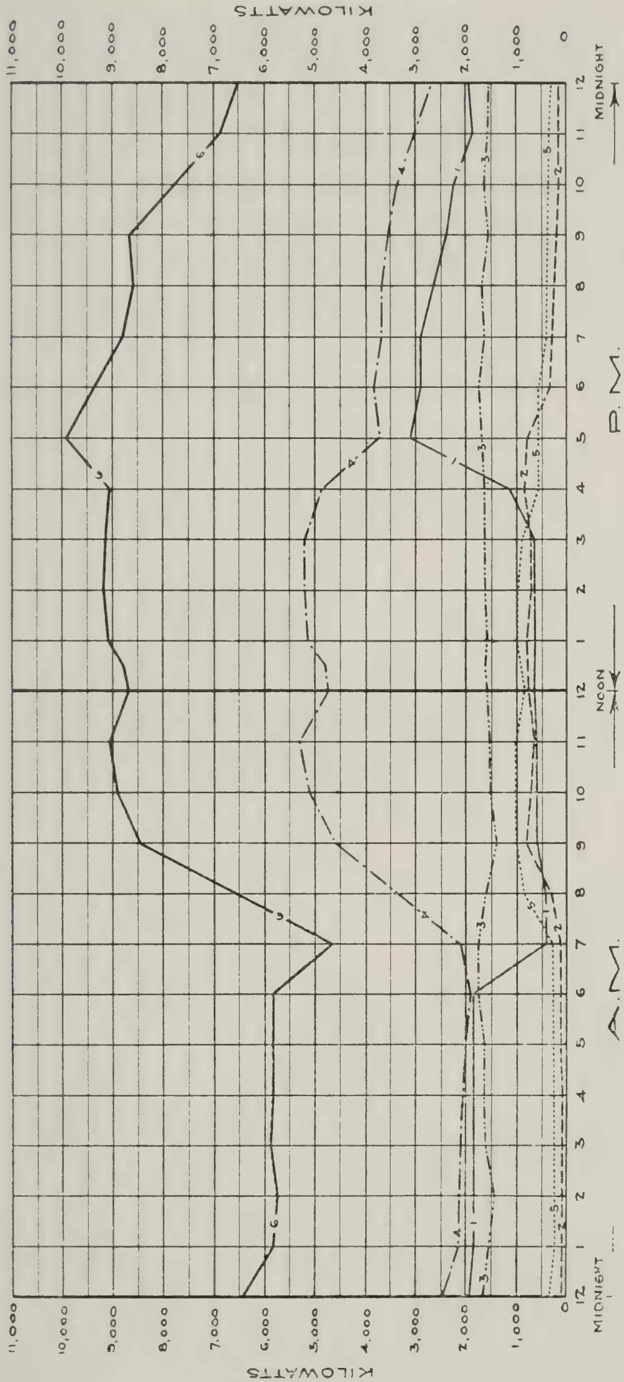

Fig. 2. Electric current requirements of the United States and the District of Columbia. Curve 1, municipal; curve 2, load where electric current is at present purchased; curve 3, steam plants to be superseded; curve 4, electric plants to be supplanted; curve 5, new buildings; curve 6, total load.

from Georgetown Reservoir to McMillan Park Reservoir. New gate houses will be needed. The pumps to pump water to the filters will also have to be increased in size or numbers, the piping at the filtration plant will have to be changed to accommodate the increased supply, and the filtered water reservoir will also have to be enlarged. The present coagulating works, located between Dalecarlia and Georgetown reservoirs, can be made to serve both for the present and increased supply.

ESTIMATES.

The estimated cost for the water supply only, project No. 3, is $3,472,600. Project No. 1, the parallel line project, will be necessarily somewhat changed. It was contemplated, as originally designed, to lead direct from Dalecarlia Reservoir to McMillan Park Reservoir, but in order to take advantage of the coagulating plant already constructed, it should lead to the Georgetown Reservoir, and thence by tunnel to McMillan Park. Additional storage is also believed to be necessary for the greater supply, and this is to be provided for at Stubblefield. As changed, the estimated cost of project No. 1 is $6,567,100; the estimated cost of project No. 2 as given in the report on the Patuxent River, to which is added the cost for the changes at the Filtration Plant, not estimated therein, is $6,215,000. The annual cost of operation and maintenance, including interest at 3 per cent on the first cost for the three projects, is as follows:

Project No. 1	$351,300.00
Project No. 2	410,200.00
Project No. 3	299,900.00

These estimates show an annual saving in favor of project No. 3 of $51,400 over project No. 1, and of $110,300 over project No. 2. These sums, capitalized at 3 per cent, represent a capital investment of $1,713,333 and $3,676,666, respectively. It appears, therefore, that at least $1,700,000 in the development of the power project could be charged to water supply and still leave a credit in favor of the combined method for increasing the water supply.

PRELIMINARY CONCLUSION.

The maximum daily draft of water already exceeds, as shown by table 1, p. 340, a safe amount for the present storage and pumping capacity. In less than eight years the average consumption will

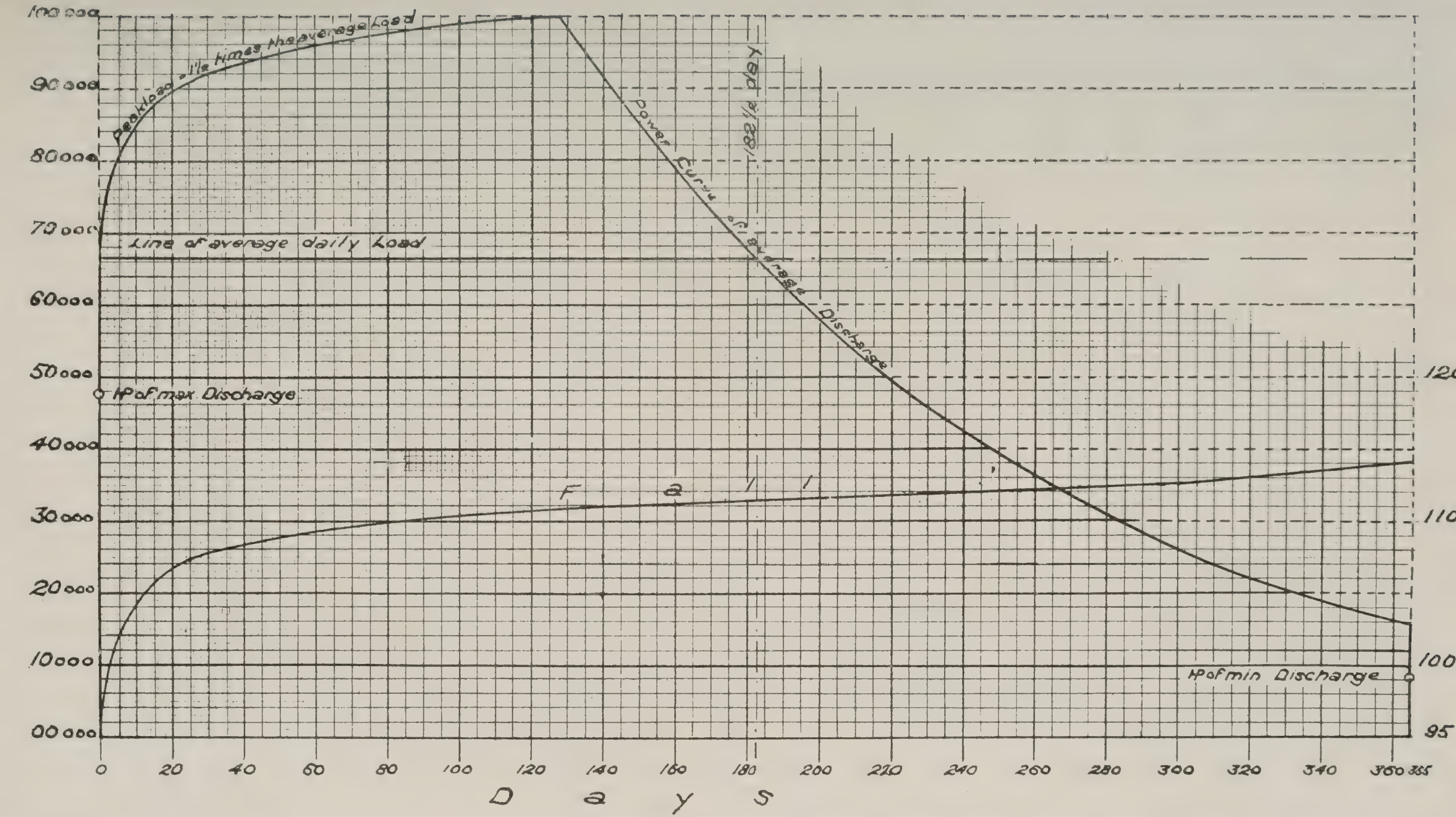

Fig. 3. Showing power that can be generated with the varying fall as the river rises from a minimum to the maximum. After the discharge reaches about 21,000 second-feet the power decreases as the discharge increases, due to decreased head, until at the maximum discharge the horsepower available is only about 47,500. The figures on the left of the diagram show horsepower developed, while those on the right show the available fall in feet at different stages of the river.

also probably exceed this safe limit. To provide for delays in construction and unforeseen contingencies, it is evident that a final decision as to the method to be adopted of increasing Washington's water supply should now be made and the work commenced without undue delay.

The preceding discussion shows that the cheapest of the three methods of increasing the water supply depends on an economical development of water power from the Great Falls. This phase of the subject will now be discussed.

POWER REQUIRED.

An attempt was made to determine the present needs for power and light, and, based on past growth, on estimates of those having charge of existing plants, and other pertinent data, a similar estimate of probable future needs was prepared. The results as to present requirements are shown graphically on Fig. 2 (page 343). The conclusions from the studies are:

First. The present average daily amount that must be available to safely meet the expected daily demands is about 8,000 kilowatts, with a maximum or peak requirement occurring between 3 and 6 o'clock p. m. of 11,000 kilowatts.

Second. That by 1937 the corresponding figures will be 17,300 kilowatts and 26,000 kilowatts.

POWER AVAILABLE.

Quantity of Water. Making due allowance for the amount needed for operating the Chesapeake & Ohio Canal and for a water supply of 101,000,000 gallons of water daily, as an average, the observed discharge of the Potomac River for fifteen years shows that occasionally the amount available for power will fall as low as 950 second-feet. By averaging the low-water discharges for fifteen years, without regard to date or day of the year in which occurring, Mr. Herschel finds that the average low-water discharge at the District line is 1,486 second-feet, allowances for the canal and water supply having been made. This amount, with the fall available, will give power to meet the average daily requirements, as will be shown later. As the discharge increases from low water, the power available also increases, and, determined in the same manner, Mr. Herschel finds that about 6,560 second-feet or more will be available one-half the average year. These results indicate the possibility, at least, of a desirable power development. It is understood,

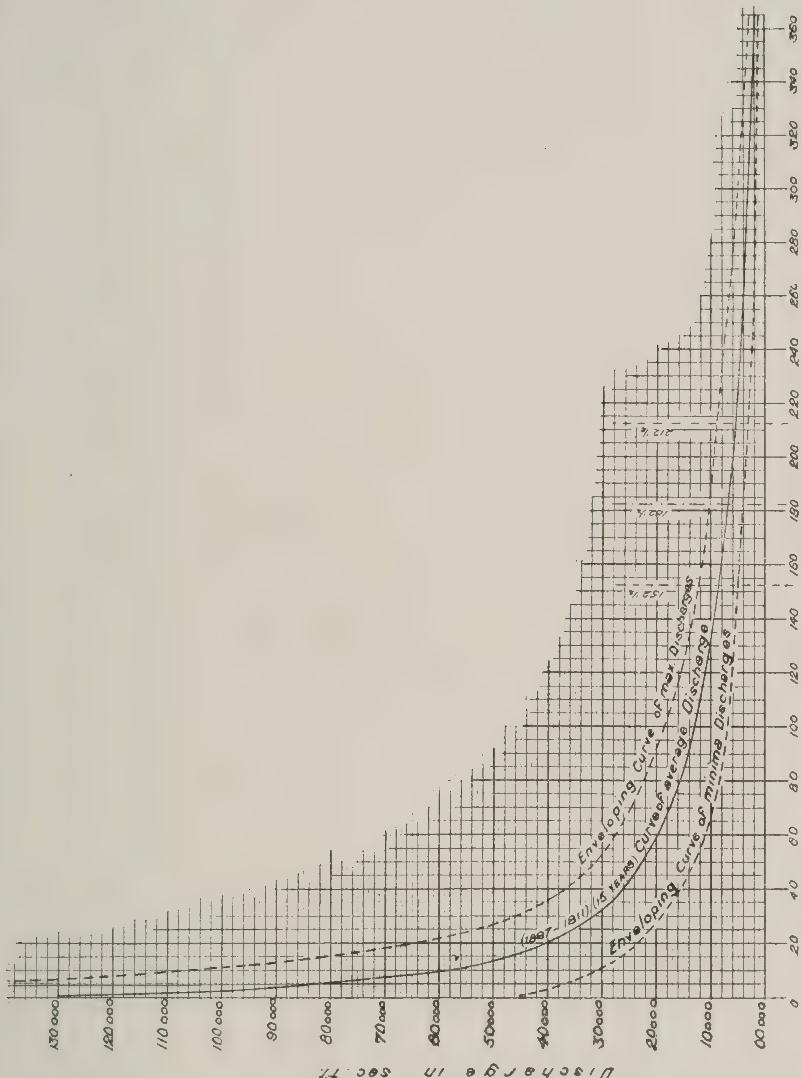

Fig. 4. Showing maximum and minimum discharges of the Potomac River in second-feet for any continuous time from 1 day to 365 days, from records extending through 15 years.

of course, that in the dry periods a steam reserve will be necessary for a portion of the time.

Available Fall. The total fall at low stages from the water surface above Great Falls to low water in the Potomac just above the Chain Bridge is approximately 150 feet. Only two methods of utilizing this fall have ever been deemed worthy of serious consideration. The first involves a head race or canal to lead the water from Great Falls to some suitable site below where it is returned to the river through the power plant located on the banks of the stream. The second method involves the construction of a dam at some point in the stream below the Falls, to create the necessary head or fall of water. Were the 150 feet of fall concentrated within a short distance of the Great Falls, the construction of a dam would not be desirable, but as this fall is distributed over a length of river of about 9 miles, such method of development may well compete with that of a lateral canal in dealing with the larger quantities of water required for economical development of power. In either case, in the present instance, it must be remembered that increased water supply must be included in any plan adopted. A thorough study was made of the plan of a long lateral canal leading from Great Falls to the vicinity of Dalecarlia Reservoir, but this plan was rejected for the following reasons:

a. The cost of the output per kilowatt hour is greater than permissible.

b. There is no equalizing basin or pondage to furnish storage for peak loads.

c. It is not economically practicable to provide for present needs with provision for future enlargement to obtain all power considered economically proper.

d. The utility of Dalecarlia Reservoir as a settling basin would be destroyed.

This leaves, then, for consideration, the construction of a dam at some point below Great Falls, and Mr. Herschel, who developed this portion of the project, recommends a dam practically where the District line crosses the river, which shall maintain the water surface at low periods at elevation 115. This solution of the problem has many advantages, among which may be named:

a. The provision for additional water supply without undue lift for the pumps.

b. The power-house and dam form practically one continuous structure, enabling the development to be made in piecemeal, if desired, without ultimate large increases in cost.

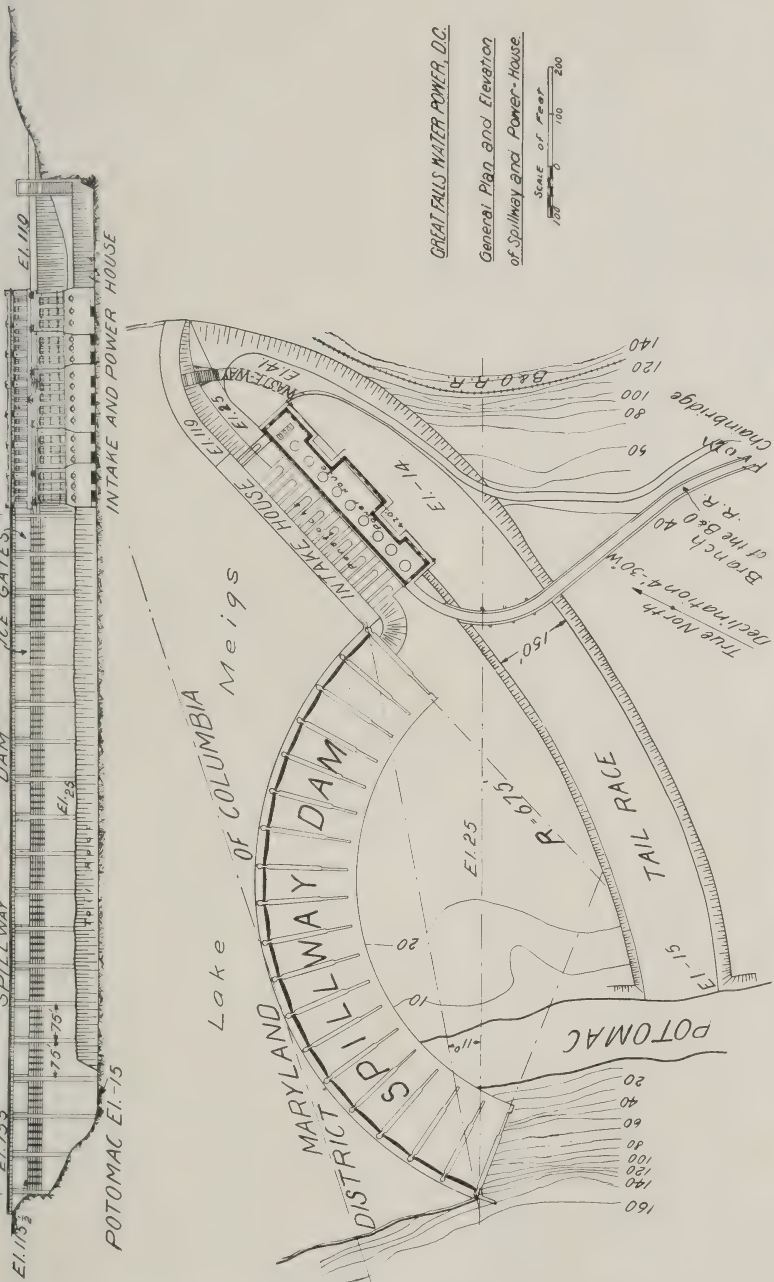

Fig. 5. General plan and elevation of spillway and power-house.

c. Lake of an area of approximately 3,000 acres permits control of the flow of the river so as to provide stored water to take care of the peak load.

d. The lake, lying immediately adjacent to the city, would be a scenic attraction and ornament to the Capital.

e. The lake would also act as a purification, a settling, or sedimentation basin for the water supply.

The elevation selected to be given to the lake surface is a compromise between the desire to obtain as high a fall as possible and the necessity of reducing overflow damages due both to ordinary flowage and backwater from floods. To reduce the latter, the crest of the dam has been made with Stoney gates, so that by raising the gates increased discharge area is given, and backwater thereby reduced. The increased opening thus possible will, Mr. Herschel states, keep the backwater at Widewater to a level not greater than that reached by the highest recorded flood, that of 1889, and during which the discharge reached approximately 470,000 second-feet.

As the elevation of the water surface above Great Falls is 150.5 and that of the lake 115, there remains 35.5 feet difference of elevation, or, allowing 2 feet as the slope of the lake surface, 33.5 feet available for power development at some future time if found necessary or advisable.

DISCHARGE AND AVAILABLE POWER.

Table 2. Available horsepower effective of the Potomac River at Chain Bridge for a peak load equal to one and one-half times the average load for the day.

Days.	Discharge at Aqueduct Dam.	Discharge at Chain Bridge.	Gauge height.	Fall.	Horsepower.	Days.
	Cu. ft. per second	*Cu. ft. per second*				
365	1,800	1,486	10.5	114.0	15,400	365
300	3,000	2,695	12.0	112.5	27,500	300
250	4,000	3,703	12.6	111.9	37,600	250
220	5,000	4,711	12.9	111.6	47,700	220
182.5	6,835	6,560	13.2	111.3	66,300	182.5
159	8,000	7,735	13.4	111.1	78,100	159
128	10,117	9,870	13.6	110.9	99,500	128
105	12,500	12,271	14.1	110.4	98,200	105
95	13,520	13,300	14.3	110.2	98,000	95
58	20,000	19,831	15.3	109.2	95,400	58
32	30,000	29,911	16.6	107.9	92,600	32
20	40,000	39,991	17.8	106.7	90,000	20
14	50,000	50,071	19.0	105.5	87,600	14
2	100,000	100,471	25.5	99.0	74,600	2
1	120,000	120,631	28.0	96.5	70,200	1

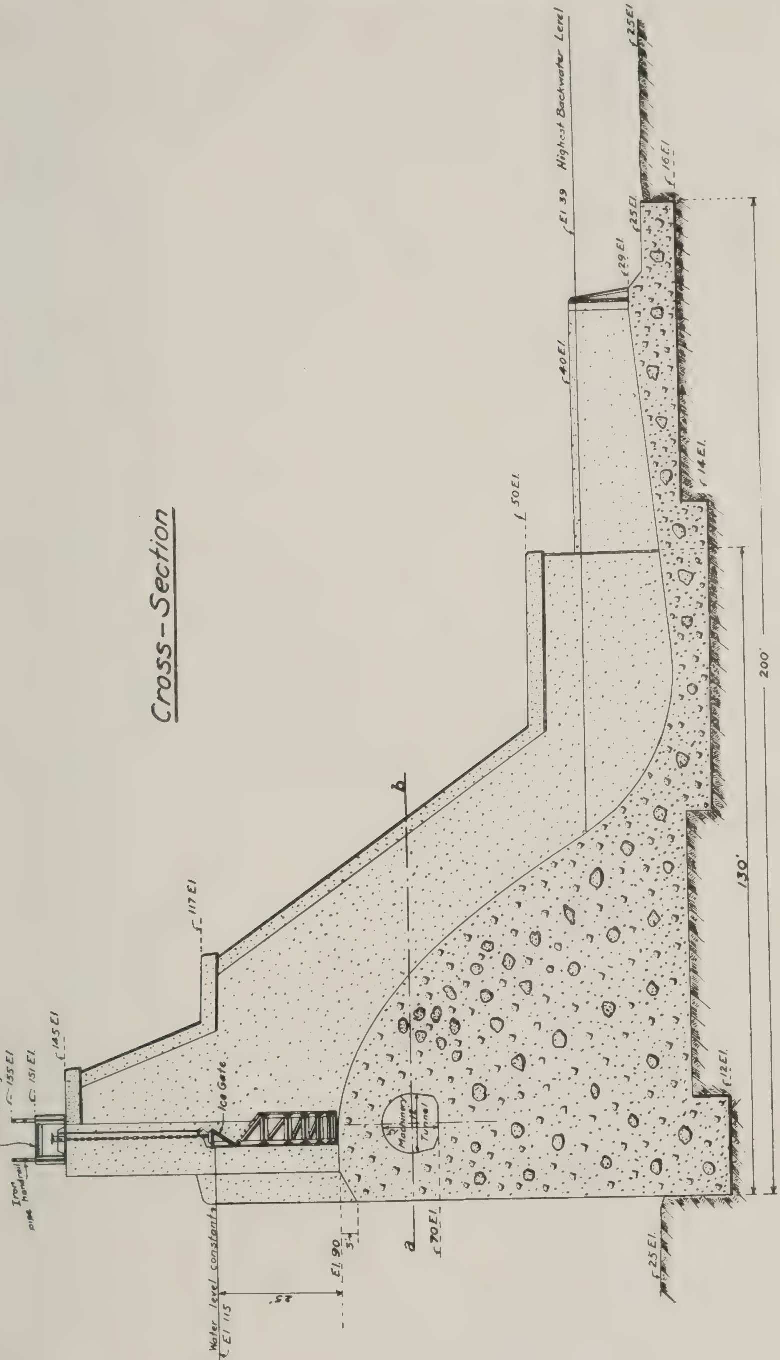

Fig. 6. Section through the spillway dam along center line of one of the buttresses that support the Stoney gates to control the water level.

The results given in the above table are also shown in diagram (Figs. 3 and 4, pages 345 and 347). Fig. 3 shows that for average years the minimum horsepower available will be 15,400, increasing with discharge up to 98,200 horsepower and then decreasing to 70,200 horsepower in high floods.

AUXILIARIES.

As indicated before, for dry years any deficiency in water power will be met from steam auxiliaries. These are already in place. The steam plant lighting the Capitol, Library of Congress, Senate and House Office Buildings, Coast and Geodetic Survey Building, etc., has a capacity of over 10,000 horsepower, and will supply all necessary deficiencies in dry years for a long period. When the limit of this plant is reached, either additional steam auxiliaries may be constructed in connection with the heating plant of any new Government building or it may prove better to develop the unused fall of 33.5 feet at Great Falls.

GENERAL DISCUSSION.

Mr. Herschel assumes as his unit of power a 12,500-horsepower turbine. Two such units will be needed to meet present requirements, including peak demands. With a third unit as a reserve, the installation will be complete for existing needs. As consumption increases more units will be needed, until for the 47,000-horsepower maximum predicted in 1937 four such units will be required, with a fifth for reserve. A power-house to accommodate this number of units should, however, be built at once, as a matter of economy; and as it is proposed as previously indicated, to provide for the increased water supply by pumping from the power lake, space for these pumps must also be provided. Table 2, before given, shows that for one-half of the average year 66,000 horsepower for the twenty-four hours is available from the river discharge. This would represent a peak load of about 100,000 horsepower, using 1½ as the ratio between the average load and peak load. It is calculated that out of a total of 438,000,000 kilowatt hours output for the average year, auxiliaries would have to furnish but 99,000,000, and would be in operation but half the year. This larger development could be obtained by extending the power-house so as to contain four additional turbine units, and in the design adopted the foundations of the power-house and the intake dam itself are arranged so that these four additional units may be

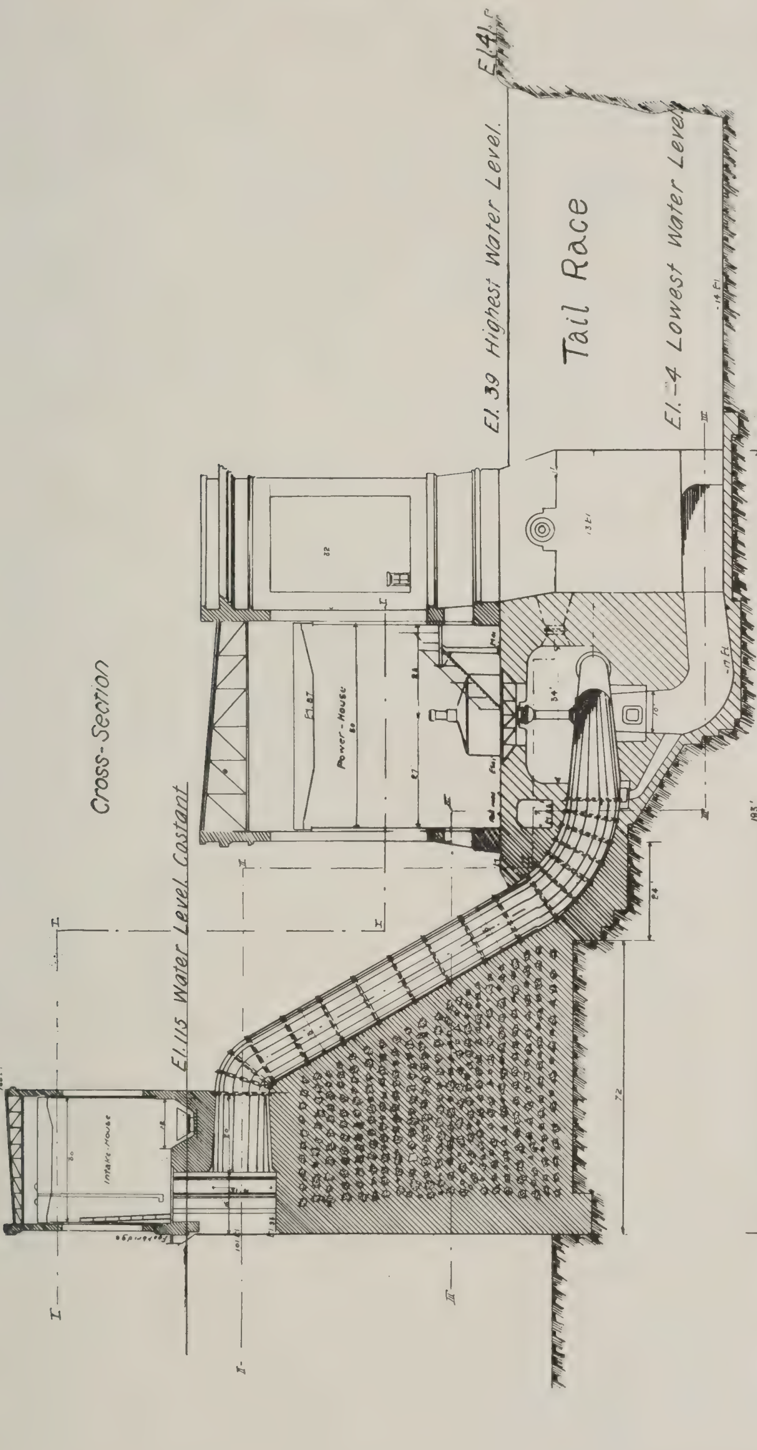

Fig. 7. Section through dam and power-house along center line of draft tube, showing one of the nine turbines in complete plan.

installed, should Congress decide so to do, either by itself or by agreement with a power company.

WORKS PROPOSED FOR POWER DEVELOPMENT.

The chief features are the dam, power-house, hydraulic and electrical appurtenances for power and turbine pumps, and pipe lines for the water supply.

The plan of the dam and power-house is shown on Fig. 5 (page 349), while Fig. 6 (page 351) gives a section of the dam through the center of one of the buttresses supporting the Stoney gates. The dam is curved in plan, but of gravity section, with eighteen Stoney gates operated from a tunnel left in the masonry of the dam. These gates, as already stated, are to provide additional discharge area in floods. A small footbridge connects the piers between the gates, and a floating caisson is provided which can be floated to and made to close any opening on the upstream side of the gate, which can then be repaired. The three Stoney gates next to the power-house have hinged tops to facilitate passing ice, drift, etc., through the dam without too much loss of water, and for the same purpose an auxiliary spillway is provided near the shore end of the power-house. The foundations of the dam are apparently excellent, but detailed information could not be obtained within the limits of available money. The dam in its general features resembles the Gatun Spillway dam of the Panama Canal. The power-house and intake dam are shown on Fig. 7 (page 353).

The proposed location is just within the District and distant about 5 miles from the Executive Mansion. This short distance permits the omission of step-up transformers and thus simplifies the electrical installation. The usual traveling crane for handling the heavy machinery is included, together with a small heating plant, store rooms, workshop, and other minor accessories. The power-house is protected by the intake dam, which is located just above it and through which pass the penstocks leading to the turbines. The intake openings are protected by specially designed racks. The intake house over the intake dam furnishes shelter to the operating force in inclement weather, protects the racks, and contains a light traveling crane. This latter may be used in cleaning out the racks or to place a stop gate over any penstock opening desired.

The turbines are of 12,500 horsepower capacity each and are di-

Fig. 8. Map showing conduits for main and branch cables and the location of substations. The latter being numbered.

rect connected to an 8,000-kilowatt generator. These latter deliver the current at 13,500 volts, which is sufficiently high to obviate, as already indicated, the use of step-up transformers. The usual switchboard, switches, bus bars, lightning, and other protective devices are provided for.

PUMPS FOR INCREASED WATER SUPPLY.

There are to be three 700-horsepower turbine pumps delivering water from Lake Meigs into a 60-inch force main leading to Dalecarlia Reservoir. The net lift is 31.5 feet. Two only of these pumps are needed at first. The subsequent disposition of this supply has already been described.

It is proposed to build the foundations for the whole power-house, but to construct at first, unless otherwise decided by Congress, only one-half the power-house. This half will contain three generating units, space for the fourth and fifth, and pumps for the water supply. The fourth unit, if installed, would at once furnish surplus power a larger portion of the year available for sale.

REDUCED ESTIMATE.

Assuming that Congress may prefer, in the first instance at least, to provide for the development of only that portion of the available power needed to supply municipal and governmental activities, a considerable reduction in first cost can be made. The unit cost of power delivered will, however, be increased, as many of the larger items of cost are the same, whichever plan be adopted.

As previously stated, the peak consumption expected in twenty-five years amounts approximately to 50,000 horsepower (47,000), requiring four turbine units, each 12,500 horsepower, and one reserve, or five in all. The power-house should be of such size as to accommodate this number of generating machines, as well as the three pumps for increased water supply. To meet demands for some years to come, three generators and two pumps will be sufficient, and these numbers only need be installed at time of first construction.

MAIN POWER HOUSE.

The reduction in first cost under the assumptions of the pre-

ceding paragraph amounts to $1,211,800, and the estimate of cost becomes:

a. Spillway dam	$2,502,000
b. Intake dam and power-house	2,951,200
c. Relocation Chesapeake & Ohio Canal, Baltimore & Ohio Railroad, electric railway	1,849,025
d. Land and water rights	1,500,000
e. Engineering and contingencies	547,775
Total	$9,350,000
Credit charged to water supply (see remarks under water supply)	1,700,000
Balance	$7,650,000

DISTRIBUTION OF POWER.

The foregoing discussion covers the project to include the generation of the electrical energy. Its distribution must now be considered.

As a matter of economy, the delivery of electrical power for any considerable distance is made at relatively high voltages. At or near the site of its use it is transformed into a current of the voltage and strength most suited to the particular case. This leads, in the distribution throughout a large city, to the selection of sub-areas, each supplied from a station receiving the energy at this higher voltage, transforming it as necessary and delivering it to immediately surrounding territory. Usually there is one main substation, which controls the distribution to the several other substations.

After much study it has been decided to adopt in the present instance five substations: Nos. 1, 2, 3, 4, and 5. The location of these substations and the boundaries of the districts covered by them are shown on Fig. 8 (page 355).

No. 1 is the main substation, and to it is led the electric energy generated at the main power-house, called "Generating station" on the figure. This main substation is located just south of the present post-office building in block 324, already in possession of the Government. Underground transmission through conduit will be used in all transmission lines, as so small a part of them is without the area in which pole lines would be undesirable or against the rules of the municipal government. To prevent interruption of service due to accidents, the conduit from the genera-

ting station to substation No. 1 is in duplicate, following different routes.

Substation No. 1, in addition to being the main substation as regards the other four, also serves the congested portion of the city. It is connected through duplicate routes with the power-house, supplying, by means of a steam plant, light and power to the Capitol group of buildings. By this interconnection this steam plant, with but few minor alterations and additions, thus becomes available as a steam auxiliary, and, having been originally designed with ample provision for increase in demands upon it, will so serve for a number of years. Further, there are enough Government-owned steam plants now in operation to supply over three times the present demand for Government power and light. Similar alterations could be made in some of these, if additional steam reserve should prove necessary before final and definite decision is made by Congress as to the course to be followed in disposition of surplus power.

Substation No. 2 controls the district west of Rock Creek (Georgetown and vicinity) ; No. 3 the district east of Rock Creek north of District No. 1, and includes the Soldiers' Home; No. 4 controls the northeast section of the city; and No. 5 the southeast section, including Anacostia.

All these substations are to be substantial, creditable in appearance, and designed to accommodate reasonably prospective growth. They will be fireproof in every respect. They were designed by Mr. Waddy B. Wood from data furnished by this office.

In solving for the usual city conditions a problem of the character of the one under study, it is evident that some limit must be placed on development considered. Usually this is taken at the substations, and this limit is so fixed in the present instance. It is to be recognized, however, that beyond this there is still a large amount of work to be done to change from existing conditions to the new ones introduced.

CONDUITS AND CABLES.

The lengths of the different sizes of cables required and of the corresponding conduits have been estimated as closely as practicable. The conductors of the cables have been taken so as to keep the loss of energy within reasonable limits. Mr. Allen, who prepared the estimates, gives figures for three different classes of insulation, viz, paper, varnished cloth, and rubber. In the esti-

mate below, the estimate for varnished cloth is taken. The estimates are:

Conduits, approximately	$500,000
Cables	850,000
Total for conduits and cables	$1,350,000

SUBSTATIONS.

As already stated, there are five substations. All are to be built of light-colored wire-cut bricks with Indiana limestone trimmings. The estimates are:

Substation No. 1	$140,318
Substations Nos. 2, 3, 4, and 5	67,650
Total	$207,968

ELECTRICAL MACHINERY.

The estimates for the electrical machinery for the main power house are included in the estimates for that structure.

The remainder of the estimate is:

Electrical installation for five substations	$641,000

This estimate includes changes in Capitol power plant to fit it to act as steam reserve and to act also as substation to the buildings served by it with necessary power from the Falls development. It also includes changes in the plant of the Bureau of Engraving and Printing to enable power to be furnished through it to the buildings served by its plant.

TOTAL FIRST COST.

Summing up the costs of the various elements, the following results:

a. Estimated cost main generating station ($1,700,000 charged to water supply)	$7,650,000
b. Estimated cost of conduit and cables	1,350,000
c. Estimated cost substations	208,000
d. Estimated cost of electrical installation	641,000
Total	$9,849,000

COST PER KILOWATT HOUR.

Estimates of cost per kilowatt hour are based on operating and maintenance charges, expected, together with allowances for depreciation, sinking fund, and interest. What should be allowed

for these various items is largely a matter of judgment. Allowing 2 per cent for interest and 1½ per cent for sinking fund, we get 3½ per cent per year for the two items. The allowance of 1½ per cent per year will return the capital in forty-three years, allowing compound interest at 2½ per cent on the sums yearly set aside. It is believed this should be satisfactory. The interest and sinking fund estimate will be then: $9,849,000 at 3½ per cent, or $344,715.

For the conduit line 1 per cent depreciation will be assumed and for the cables 4 per cent. For the electrical machinery 4 per cent per year depreciation will be taken and on the substations 1 per cent per year.

For operation and maintenance of the whole plant, $100,000 is estimated as ample.

Depreciation of machinery, structures, etc., of the main power house, using 1 per cent on masonry, 2 per cent on steelwork, and 3 per cent on machinery, is given by Mr. Herschel as $57,149. Using the assumption above given for conduits, cables, electrical machinery, and substations, the following results:

Conduits, 1 per cent of $500,000	$5,000
Cables, 4 per cent of $850,000	34,000
Electrical machinery, 4 per cent of $641,000	25,640
Substation buildings, 1 per cent of $208,000	2,080
Total estimate of depreciation	$66,720

Adding Mr. Herschel's estimate, the total for depreciation becomes $123,869.

Collecting the above estimates of yearly cost, there results:

Interest and sinking-fund charge	$344,715
Depreciation	123,869
Operation and maintenance	100,000
Total	$568,584

Assuming that the plant will not be ready for operation for, say, five years from January 1, 1913, and estimating conservatively that the average demand on the power plant by that time will be only 10,000 kilowatts, the yearly consumption will amount to 72,000,000 (24 by 300 by 10,000) kilowatt hours. This gives the cost per kilowatt hour of 8.04 mills. This is a very reasonable cost and it is believed that none of the steam plants now used for production of electrical energy for governmental uses can produce this energy as cheaply, the same basis of calculation being used.

The average price to the Government and the District for all current consumed was 2.011 cents per kilowatt hour for 1912. If interest and sinking fund charges are omitted, as seems to be the case in some of the costs given by Captain Bain, the cost per kilowatt hour is at once reduced from 8.04 to 3.1 mills. It is evident that no steam plant can approach this figure in cheapness of production.

It seems proper to remark here that for a part of the drier years the steam reserve provided for in the Capitol steam plant will have to be called upon, but this in no wise detracts from the advisability of the project, as on the average additional power from this plant will be needed on less than sixty-five days in the year, and the amount varying from 0 to 14,000 kilowatts, the latter only on the driest day. The kilowatt hours needed from this plant will average not to exceed 4,000,000 kilowatt hours per year. The cost of furnishing this at the Capitol power-house may be taken at 15 mills, making due allowance for losses. The average cost for the year would be 3.7 mills or 9.05 mills, depending on whether interest and sinking fund is or is not included.

ESTIMATES.

Intimately connected with the cost to the General Government of development herein submitted will be the expenditure necessary to acquire the necessary land and water rights, these latter including the power rights of all private or corporate parties. Moreover, the General Government itself is already part owner of the lands and rights in question.

The only means by which all these rights can be acquired by one agency at reasonable cost will necessarily be, therefore, one possessing the right of eminent domain. This right inheres in the General Government in its own territory, and in the various States it has been given right to acquire all property necessary for its needs by purchase or condemnation. It is taken as a settled fact, therefore, that the United States may exercise the right of eminent domain to acquire such additional land and rights as it may need for its purposes.

For the complete, or larger, development—that is, for one of

67,000 horsepower average daily load, the estimate of cost is as follows, as made up by Mr. Herschel:

a. Spillway dam	$2,502,000
b. Intake dam and power-house	4,163,000
c. Relocation of Chesapeake & Ohio Canal, Baltimore & Ohio Railroad and electric railway	1,849,025
d. Land and water rights	1,500,000
e. Engineering and omissions	585,975
Total	$10,600,000
Credit charge to water supply	1,500,000
Balance	$9,100,000

Making the usual allowances for interest, depreciation, sinking fund, operation, and maintenance, there results as the cost per kilowatt hour 2.3 mills for the estimated average annual output. The estimated steam auxiliary cost of output will bring this cost up to about 3.5 mills per kilowatt hour for the year, taking into account the relative amounts of steam and water power used. The results are very favorable, and if the United States desires to create such a large development and sell the surplus power the outlay could readily be returned to the Treasury within a reasonable period and profit made at the same time. The delivered costs in all the cases would, of course, be higher than here given, as the figures represent only the costs at the power-house switchboards.

The costs per kilowatt hour just given are sufficient in themselves to indicate strongly the advisability of the United States undertaking the work under consideration. Captain Bain gives further light on this. He estimates that the saving in 1912 to the General Government and to the District would have been $666,000 if current had been free of cost. The above calculations show that this current—in fact, a much larger amount of current—could have been furnished from the proposed plant for $568,584, indicated a saving of $97,416 annually. This, capitalized at 2 per cent, would represent $4,870,800. Against this, however, must be placed the cost of local distribution and of changing existing plants so as to enable them to take the Great Falls power. No detailed estimates of this cost have been made, but it may be safely stated that such cost will be only a small fraction of this sum—again giving a result favorable to the project.

SUMMATION AND CONCLUSION.

The following tabular statement of estimates of cost is here given for comparison:

Project No. 1 of this Report.

Estimated cost increased water supply from Potomac River, parallel line project; no power development feasible	$6,567,100
Annual cost of operation, including interest on first cost	351,300
Annual cost of operation, excluding interest	154,300*

Project No. 2 of this Report.

Estimated cost increased water supply from Patuxent River; no power development feasible	$6,215,000
Annual cost of operation, including interest on first cost	410,000
Annual cost of operation, excluding interest	223,700*

Project No. 3 of this Report.

Estimated cost of increased water supply from power project, increased by $1,700,000	$5,172,600
Annual cost of operation, including interest on first cost and increased by 3 per cent of $1,700,000	350,900
Annual cost of operation, excluding interest	115,700*
Estimated cost of development of power for General Government and municipal needs, less $1,700,000	9,849,000
Annual cost of operation, including interest and sinking fund, approximately	568,584
Cost per kilowatt hour of current delivered — mills	8.04
Annual cost of operation, interest and sinking fund not included	281,018†
Cost per kilowatt hour of current delivered — mills	3.1

These conclusions follow: 1. If increased water supply alone is considered, either of projects Nos. 1 or 2 will furnish a satisfactory solution, with but little choice as to quality and security of supply. The slight advantage of the Patuxent River (No. 2) project is believed to be outweighed by the less cost (cost of operation included in the comparison) of No. 1, the parallel-line project of the Potomac River. Therefore, if the power project is not adopted, this latter project is recommended for immediate adoption.

By adopting the power-plant project as herein proposed—and it is beyond question the most satisfactory solution, for governmental purposes, of any suggested—the total expenditures for first cost

*These are the resulting estimates of cost of annual operation and maintenance, including the present aqueduct, etc., that would have to be met by annual appropriation.

†The annual appropriation for operation of the power plant installation will amount to $100,000, more or less.

will be $15,021,600, with an estimated total annual cost of operation and maintenance of $215,000, including both power and water supply.

For this, however, there is obtained a safe and sure water supply, capable of indefinite expansion as needed at very small cost; a power plant that, from the savings effected, will return its cost within a reasonable period and also provide interest at 2 per cent. It provides opportunity for revenue from sale of power under conditions at which no one could take exception, the water power being created entirely by the General Government and the means thereof lying wholly within its domain. It should be noted that these results are obtained on consumption as estimated at time of completion of plant and that as the consumption increases, as it undoubtedly will, the corresponding savings will be increased.

There are other advantages to this plan that may recommend it to favorable consideration. It provides a lake adjacent to the city the natural beauty of which is enhanced by the scenic features of the Potomac River. The upper part of the Great Falls are undisturbed and their attractiveness undiminished at any time of the year. As an added attraction to visitors it will be worth much, and as a measure of conservation many thousands of tons of coal will be saved to the nation and the people. Its value in these ways can hardly be estimated in dollars and cents.

In view of all that precedes, the writer recommended, first, that project No. 3 of this report be adopted by Congress; second, that other projects for increased water supply be dropped from consideration; and, third, that a first appropriation be made in the pending District bill, or as a separate measure, of $3,000,000, to be expended under the direction of the Chief of Engineers in accordance with this project, for preliminary work, acquirement of land and water rights by ordinary methods of purchase or by condemnation proceedings, when, in the opinion of the Chief of Engineers, such course is deemed advisable, and for all other necessary purposes, $1,000,000 of the sum above mentioned to be applied to construction needed for the increased water supply as described in the project, and that provision be made in the appropriation for contract authorization for the remainder needed to complete the project, to be paid for as appropriations may be made from time to time by law.

Book Reviews

A TEXT-BOOK ON FIELD FORTIFICATIONS. By Col. G. J. Fiebeger, United States Army. Third edition, John Wiley and Sons, 1913. Price, $2.00 net.

As stated by the author in his preface to the first edition, this book was written for use in the course of instruction in the United States Military Academy. Having been prepared for a special purpose, it is but natural that it should differ somewhat from the ordinary works on the same subject. Siege works have been omitted, being taught in connection with military mining, while chapters on the passage of rivers, military demolitions, and communications have been included.

The present edition bears little resemblance to the first edition; in fact, as stated by the author, it has been entirely rewritten. In view of the experiences of the South African and Manchurian campaigns and the consequent correction of views as to the part played by fortifications in war, radical revisions of existing text-books on field fortifications have become necessary, and this has been most effectively accomplished by the author in the third edition of his work.

The fundamental principles governing the employment of field fortifications in campaigns are clearly and concisely set forth, and fully supported by examples from the most recent wars, as well as by pertinent references to the United States Field Service Regulations, United States Infantry Drill Regulations, and to official reports of the South African and Manchurian campaigns.

The text of 134 pages is followed by three appendices and 27 plates, the latter of excellent workmanship.

It is not too much to say that he who shall have mastered this little volume will have a most thorough and correct knowledge of field fortification in its relation to modern warfare.—J. E. K.

RUSSIAN REGULATIONS FOR FIELD FORTIFICATIONS FOR OFFICERS OF ALL ARMS, 1910. French translation by Chef de Bataillon A. P. A. Painvin. Published by Henri Charles, Lavanzelle, No. 10 Rue Danton, Boulevard St. Germain, 118, Paris, France. Price, 50 cents.

These regulations complete and supplement corresponding regulations concerning the same subject for infantry, field artillery, and engineer troops.

The original regulations are divided into four parts:

a. Field fortifications; b. Military ways or communications; c. Telephones; d. Bivouacs.

The translation at hand reproduces at considerable length the substance of "a" as far as concerns the infantry, omitting what specially concerns the field artillery only. Extracts of general interest from "b," "c," and "d" have also been reproduced.

The book contains 91 pages, including about 72 illustrations; the dimensions are reduced to the metric system.

The intention of the regulations is to explain all field construction work which the infantry and artillery will be called upon to execute along or under supervision of the engineers, or in which they may assist the engineers. They form a kind of hand-book for field service on these subjects:

Chapter I deals with the defensive. The subject of trenches is first discussed and the various types illustrated, showing how the cross-section varies under different conditions and how to protect a line of trenches from enfilade fire or localize the effect of shells.

Various kinds of loopholes, headcover, splinter-proofs, and communicating trenches are then described and illustrated, and the question of drainage is discussed.

The Russians evidently have three different types of machine guns, and the details of emplacements for each are given.

The supporting points or redoubts described follow the general trend of those evolved as a result of the Manchurian War. Type forms are given, but their application of the special contours of a position are shown. Under this head are also descriptions of the obstacles, dummy trenches, and other accessories of the redoubts.

Chapter II deals with the rôle of entrenchments on the offensive, including the attack of fortified and unfortified positions and night attacks. It then gives the general conditions and the special conditions to be fulfilled in the defense of a place or position, as to obstacles, supporting points, secondary lines, etc.; then describes the method of procedure of the troops assigned to construct different parts of the line and the methods of defending the same, including both personnel and matériel.

Parts II, III, and IV deal briefly with the subjects of the passage of streams, the use of military telephones, and expedients to use in bivouacs to make the men comfortable.

There is nothing really new in the book for one who has followed closely the recent trend of field fortifications, but few expositions concerning the placing of trenches and supporting points have been as clear and definite as this one.—W. D. C.

A Staff Officer's Scrap Book. By General Sir Ian Hamilton, G. C. B. Second edition published by Longmans, Green and Co., of New York and London. 444 pages and 42 maps and sketches. Price, $2.10 net.

A wonderfully well written book by a trained eye-witness who traveled with Kuroki's army from the battle of the Yalu until a few days before Mukden. To great ability as a writer, Ian Hamilton adds the enthusiasm of youth tempered with long years of ex-

perience in observing and reporting events on the battlefield. Not only is he a highly trained military observer, but a close student and deep thinker on world affairs and the causes that control those affairs. He has a great fund of general information gathered from both observation and reading with which he illuminates his story, thus making his book one of the comparatively few that can be read with equal profit by the soldier and the civilian.

Not only is Sir Ian Hamilton a deep and independent thinker, but he has the courage and, perhaps we should add, the rank which enables him to state his opinions even when, as occasionally happens, those opinions are severe, not to say caustic, criticisms of the English War Office and even of the English Government itself. It is not conceivable that any American officer would dare criticise his own Government and War Department as Sir Ian Hamilton has criticised, at times, the English Government and the English War Office, and yet those very criticisms, written on battlefields during a great war that has changed the course of world affairs, should be of the greatest possible value to England.

To Americans the book is peculiarly valuable at the present time, when too many are inclined to look upon anything teaching military preparedness as abhorrent to twentieth century civilization. Sir Ian Hamilton has no such delusions; to him the idea of disarmament, or even the serious reduction of armaments, by any civilized nation to-day, is so fraught with danger that he can hardly discuss the subject. Indeed, to him, disarmament by any European or American nation just when other nations on the Asiatic continent with totally different ideals in morals, religion, and practically everything else, are barely beginning to realize the power of arms and ammunition, means national suicide.

And to any Caucasian who stops to think of the hundreds of thousands of policemen, sheriffs, judges, and the hosts of attendants on each, that are employed to keep his own countrymen at peace, it ought not to be necessary to speak more than once to show him the absurdity of expecting to get our rights in the world, among other nations so totally different in all their ideas, unless we are prepared to back with men and arms every just demand.

To the military man, one of the most important lessons emphasized in the book is the necessity for quick, determined initiative. One does not need to read half way through the book to discover that the Russian soldier was not only as stubborn a fighter as the Japanese, but that he was probably more indifferent to danger on an average than the Japanese themselves and yet the Japanese invariably won. Where numbers were about equal, the success of the Japanese was due to two things: Initiative and the ability to make tremendous marches over difficult country. It is true the Russian was able to make those same long marches, but he lacked the initiative and daring so necessary to final success in such movements. The Russian was too prone to remain in his chosen lines and await the attack and, evidently, the Japanese counted greatly

upon this Russian trait, because time and again they dispersed their numbers in a way that would have been frightfully dangerous had the Russians possessed the same initiative as the Japanese.

One other point which Sir Ian Hamilton emphasizes and which must not be lost sight of is, that to-day it is probably more necessary than in former years to close with the enemy to within striking distance with the bayonet at the earliest possible moment. This, he shows, must be done through night attacks and surprises which necessitate, as before mentioned, great initiative coupled with the ability to make long marches under difficult circumstances. He also demonstrates that the general who counts on winning a battle in the future by sitting down in trenches and maintaining a long range duel will be beaten; a fact that is being emphasized just at present in the war between Turkey and the Balkan allies.—A. A. F.

Treatment of Gun Emplacements*

I take a little gravel,
And I take a little tar;
With various ingredients,
Imported from afar,
I hammer it and roll it,
And when I go away,
I think they have a 'placement
That will last for many a day.

But——

I must come with picks and smite it,
To lay a water main;
And then I call the workmen
To put it back again.
To run a hoist or cable
I must take it up some more,
And then I put it back again,
Just where it was before.

I must take it up for conduits
To run the telephone;
And then I put it back again
As hard as any stone.
I must take it up for wires
To feed the 'lectric light,
And then must put it back again,
Which is no more than right.

Oh! the platforms full of furrows,
There are patches everywhere;
I would like to swear about it
But 'tis seldom that I dare.
They are very handsome 'placements,
A credit to the Corps!
We are always digging of them up,
Or putting down some more.

—Anonymous.

*(Paraphrased and slightly remodeled from a brilliant Western poet by an exasperated dauber of cement.)

Editorial Notes

Award of Prizes

The School Board of the Engineer School has awarded the four prizes offered to Junior and Assistant Engineers for the best articles published in the PROFESSIONAL MEMOIRS during the year 1912, and has forwarded checks for the amount of the prizes as follows:

First Prize, $50.00, to Mr. E. J. Duffies, Assistant Engineer, United States Engineer Department, for his article in No. 17 entitled, "Description and Cost of Concrete Superstructures for Breakwaters at Harbor Beach, Michigan."

Second Prize, $25.00, to Mr. J. M. Pratt, Assistant Engineer, United States Engineer Department, for his article in No. 14 entitled, "Dredges and Dredging in the Mobile District."

Third Prize, $15.00, to Mr. J. D. DuShane, Assistant Engineer, United States Engineer Department, for his article in No. 18 entitled, "Hydraulic Dredges and Dredging in the Improvement of the Upper Mississippi River."

Fourth Prize, $10.00, to Mr. C. H. Tisdale, Junior Engineer, United States Engineer Department, for his article in No. 13 entitled, "Treatment of the Foundation for the Power House and Dam, Hales Bar, Tennessee River."

Although, when the offer of these four prizes was originally made, it was stated that awards would be made by three assistant engineers, not competitors, it has been found impracticable to do so, thus making it necessary for the School Board to make the awards.

In this connection, the editors wish to again call attention to the fact that the same prizes are offered for the best articles published during the year 1913, the competition being open, however, to all subscribers, with the exception of officers of the Corps of Engineers with more than ten years service in the Corps.

Appreciation and Advice

WAR DEPARTMENT,
Washington, March 3, 1913.

Dear General BIXBY:

* * * I have a very high admiration for the ability that you have shown in making the recommendations which have come to

me on all of the varied and intricate problems of your office; and it has been an even greater satisfaction to realize that in all of these problems, involving such great financial responsibility, I could always rely upon your advice as being actuated solely by the highest traditions of your Corps.

The men of your Corps have always shown themselves equal to any professional or financial responsibility. I want to remind you, however, that a quasi political responsibility—not at all in a partisan sense—is now being more and more thrust upon you. In dealing with rivers and harbors Congress is too apt to represent merely the sum of local viewpoints. Too often does it wholly fail to look upon our problems from the national standpoint. At the same time, the necessity and importance of that national viewpoint is constantly increasing. The same era of combination and consolidation which has swept through our railroads, is turning the problems of our water routes from local to national problems. It is upon the Board of Engineers that the duty must fall of bringing this viewpoint to the attention of Congress If they are not national, no one will be. They are not only the advisers of Congress, but the advisers of the Executive; and from all these circumstances there grows out a duty for them to have the broadest possible perspective and the most farsighted view. I think you realize this; but I hope you will constantly keep it in mind in the future, because I know of no permanent body of men upon whose shoulders rest more of the Nation's responsibilities than the Corps of Engineers.

* * * * * * *

Faithfully yours,

Henry L. Stimson.

Brig. Gen. William H. Bixby,
War Department.

Rules for Authors

The publication of many articles submitted to the Professional Memoirs is delayed because of the incompleteness of the data, drawings, and photographs submitted. It is believed by the editors that a great deal of this delay, together with considerable of the extra work done in this office, can be obviated without any extra work on the part of the authors if certain rules hereafter given are followed.

In submitting these rules the editors do not wish to deter anyone from submitting an article because his drawings or photographs do not meet the conditions set forth herein, for the reason that if he has not the time to arrange them properly but will submit the article, the other necessary work will be done in the editorial office, providing, of course, that the article is a good one; but

where drawings or tracings must be made, or photographs be taken, especially for the purpose of the article, they should conform to the following rules:

PREPARATION OF ARTICLES.

Let the title name the subject of the article and be as short as possible; in fact, the shorter the better.

The first paragraph should give the location of the work under discussion or a very brief general description of the appliance, tool, or subject-matter of the paper.

This should be followed with what history it is intended to relate, after which the subject treated of will be discussed in as much detail as the writer deems desirable.

Ample space should be given to the description of articles and methods, remembering that what may be very clear or common to the writer may not be at all so to the reader. This applies particularly to the writer with much practical experience on construction work or in the field but with little or no practice in writing up his work. If there is no repetition, the article is not apt to be too long. Articles should always be illustrated fully with drawings and photographs. If there be too many the editor can easily omit those not desired, but if there are too few it causes no end of trouble to get the required numbers, often resulting in serious delay that is apt to destroy largely the timeliness of the article.

PREPARATION OF DRAWINGS.

Whenever an original drawing is made for use in the Professional Memoirs, or whenever another drawing is traced for that purpose, it should, so far as possible, conform to the following requirements as regards outside dimensions of the drawing, thickness of lines, heights and widths of letters, etc.

If the drawing is intended to be reproduced by a line cut, it should be arranged so as to reduce to 4 inches by 7 inches for a full-page cut and ordinarily arranged to be viewed from the long side. If, when reduced, the cut will be 4 inches wide and less than 6 inches in length, it should be arranged to be viewed from the narrow side. If it is intended that the drawing shall be lithographed, it should be drawn so as to reduce to 7 inches in width, the length being immaterial so long as it does not exceed 26 inches (reduced); though it is better to not have it longer after reduction than 20 inches and always to be viewed from the long side. The

above rules avoid making lithographs that must be folded from the top down as well as lengthwise. In submitting drawings, the original tracings or photographs of the same should be sent with the article.

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At the end of an article the author should give his full name (with all initials written out) together with the date of his birth, because all articles that appear in the PROFESSIONAL MEMOIRS are indexed and cards made out descriptive of the same by the Congressional Library, whose rules require the full name and date of birth of the author. Finally, for use of the MEMOIRS, the author should give his full official title, name of work engaged on, if any,

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Errata in No. 20

The exponent of the fraction in top line, page 145, should be $\frac{2}{3}$ instead of $\frac{3}{2}$.

The word "left," which ends the first line of the caption to Fig. 1, page 33, should be "right."

The words "Fig. 1," at the end of line 10, page 184, should read "Fig. 4."

Errata in This Number

On Plate 2, opposite page 250, near the lower left-hand corner, the time of maximum northbound tide should read "the VII lunar hour" instead of "the XII lunar hour."

It should be noted that the part of the curve in the middle of Plate B, page 253, below the line is the discharge for southbound currents and that above, the discharge for northbound currents.

In the formula, page 265, the second term of the equation, *only*, is divided by 4.

On page 278, 4th paragraph, seventh line, there should be a period after the word "kills;" the word "under" should be omitted; the word "the" begun with a capital; and the words "was therefore made" inserted immediately after the word "assumption."

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(2) American Machinist.
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(4) Canadian Soc. of Engineers. Trans.
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(13) Engineering Magazine.
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(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
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(19) Journal of Franklin Institute.
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(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
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(27) Scientific American.
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(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institution of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review. London.
(80) Journal, American Society of Engineering Contractors, N. Y.
(82) Journal, New England Water Works Association, Boston.
(83) National Waterways, Washington, D. C.

*Now combined under title: Concrete-Cement Age.

BANK PROTECTION.

Concrete bank protection for deep rivers. B. Okazaki. (14), March 13, 1913. D. I.—Methods and costs of building shore protections of the Upper Mississippi river. C. W. Durham. (12), April 9, 1913. D. I.

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Problem in beach reclamation and a prize for its solution. (14), Feb. 20, 1913. D.

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Concrete superstructure over the detached breakwater, Michigan City Harbor, Indiana. G. A. M. Liljencrantz. (12), April 9, 1913. D.—Costs of breakwater construction at Lincoln Park, Chicago. (12), April 2, 1913. D.—Design and construction of breakwaters. F. L. Smith. (Surveyor and municipal and county engineer.) Jan. 10, 1913. D.

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Colorado River siphon. G. Schobinger. (21), March, 1913. D. I.

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Marne ship canal. F. B. Mann. (15), Feb. 22, 1913. D.—Scientific management in concrete work. D. J. Hauer. (39), Jan. 15, 1913.—Shearing strength of joints between old and new concrete as shown by tests. J. R. Nichols. (Harvard engineering journal), April, 1913.—Some experiences with concrete piles in Chicago. N. Jensen. (14), Feb. 27, 1913. D.—Some experiments with mortars and concretes mixed with asphaltic oils. A. Taylor and T. Sanborn. (21), March 1913. D.—Some tests of oil mixed concrete with particular reference to permeability. (12), April 16, 1913.—Steel forms for concrete construction. (3), March 20, 1913.—Unit method of reinforced concrete construction. J. E. Conzelman. (45), April 1913. D. I.—Waterproofing concrete as an engineering problem. M. Tech. (Journal Engineers Society of Pennsylvania), Jan., 1913. I.

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Tidal movements in East River, New York. W. M. Black. (30), May-June, 1913. D.

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Arrowrock dam, Boise irrigation project. (15), Feb. 22, 1913. D.—Big Meadows dam to be remodeled. (15), March 29, 1913.—Bill to provide for State water conservation and State control of dams in Pennsylvania. (14), April 3, 1913.—Boring and grouting a fissure foundation beneath an embankment dam. D. W. Cole. (15), March 29, 1913. D. I.—Construction of Ocoee River dam. (39), Jan. 15, 1913. I.—Dam and reservoir of a German water users association. K. C. Grant. (15), April 12, 1913.—Deerfield river hydroelectric development. W. O. Rogers. (Power), Feb. 25, 1913. D. I.—Failure of the Tainter gates at Black River Falls. (15), March 29, 1913. I.—Failure of navigable pass No. 26, Ohio River. (30), May-June, 1913. D. I.—High dams of great length. R. Ryves. (Concrete institute, transactions and notes), vol. 4, pt. 2, July, 1912. D. I.—Making a cut-off wall by grouting fissured rock, Lahontan dam. D. W. Cole. (14), April 3, 1913. D. I.—Methods and cost of constructing a small concrete dam. F. G. Wood. (12), March 26, 1913.—Notable reservoir for flood control in Germany. (14), April 3, 1913. D. I.—Rockfill diversion dam with concrete core wall, Minidoka irrigation project, Idaho. P. M. Fogg. (12), April 9, 1913. D. I.—Solid timber arch dam. R. A. Lindquist. (15), March 29, 1913. D. I.—Solid timber arch dam. R. A. Lindquist. (49), March 29, 1913. D. I.—Tests at Lahontan dam of electric shovel. (12), Feb. 26, 1913.—Twin falls, Oakley irrigation project. A. M. Korsmo. (14), March 13, 1913. D. I.—Unique construction methods and devices employed at lock and dam No. 1, Mississippi river improvement. (12), March 19, 1913. D. I.—Great Falls power. W. C. Langfitt. (30), May-June, 1913. D.

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Note sur un barrage mobile automatique etabli a l'embouchure de la Sevre Nantaise. E. Pasquier. (1), Nov.-Dec., 1912. D.—Failure of navigable pass, Dam No. 26, Ohio River. (30), May-June, 1913. D. I.

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Large steel derrick for government work. (15), March 1, 1913. I.

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Hydraulic fill dike construction using turbine driven centrifugal pumps. (12), Feb. 26, 1913. I.—Miami and Erie canal feeder reservoirs during Ohio floods. (15), April 5, 1913. I.

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Freemantle graving dock: Steel dam construction for north wall. J. F. Ramsbotham. (21), Feb., 1913. D. I.

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Fire-proofing. R. L. Humphries. (Concrete Institute, transactions and notes), Dec., 1912. D. I.

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Albany filtration plant during the flood. (15), April 5, 1913. I.—Control reservoirs and the Dayton flood. (27), April 19, 1913.—Effect of the March floods on the Terre Haute waterworks. D. R. Gwinn. (14), April 10, 1913. I.—Effects of the flood in Indiana. (15), April 5, 1913. I.—Effects of recent flood on New York streams. R. E. Horton. (15), April 12, 1913. D. I.—Flood damage along New York State barge canal. (14), April 3, 1913.—Flood damage at Sidney, Ohio. W. R. Blake. (14), April 10, 1913.—Flood devastation at Dayton, Ohio. April 12, 1913. D.—Flood disasters in Ohio and Indiana. (14), April 3, 1913.—Flood effects at the works of the Indianapolis Water Company. C. H. Hurd. (14), April 10, 1913.—Flood of March 22d, 1912, at Pittsburgh, Pa. K. C. Grant. (21), Feb., 1913.—Flood of March 25th, at Akron, Ohio. (14), April 10, 1913. D. I.—Flooding of the Albany filtration plant and previous high floods at Albany, N. Y. W. Greenlach. (14), April 10, 1913. I.—Floods in New York State and the work of the State department of health. (14), April 10, 1913.—Floods in the Ohio and Mississippi rivers. (14), April 10, 1913.—Miami and Erie canal feeder reservoirs during Ohio floods. (15), April 5, 1913. I.—Mississippi floods. C. A. Zander. (27), April 5, 1913.—Notable reservoir for flood control in Germany. K. C. Grant. (14), April 3, 1913. D. I.—Notes on the Dayton flood. (15), April 5, 1913. I.—Notes on the flood. (15), April 5, 1913.—Notes on the floods in Ohio and the related work of the Ohio State Board of Health. W. H. Dittoe. (14), April 3, 1913.—Ohio Valley floods. (14), April 3, 1913. D. I.—Recent floods at Columbus, Ohio. J. Griggs. (14), April 10, 1913. D. I.—Recent great flood. April 12, 1913. I.—Recent high water in Pennsylvania streams. (14), April 10, 1913.—Sacramento River flood control. F. H. Tibbitts. (12), April 9, 1913. D. I.—Tornado and flood damage at Terre Haute, Indiana. M. A. Howe. (14), April 10, 1913. I.—Unprecedented floods in New York State. R. E. Horton. (15), April 5, 1913. I.

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Freemantle graving dock; Steel dam construction for north wall. J. F. Ramsbotham. (21), Feb., 1913. D. I.

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Foreshore protection by reinforced concrete groins. (14), April 3, 1913. D.

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Slinging and making hitches. J. Riddell. (Power), April 22, 1913. D. I.—Slings and hitches for handling machinery. J. Riddell. (14), April 17, 1913. D. I.

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Pacific. H. M. Chittenden. (21), Feb., 1913.—Typical quay and pier construction for the new port works of Portland, Oregon. (12), March 5, 1913. D.

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Appalachian hydroelectric development. W. O. Rogers. (Power), March 25, 1913. D. I.—Deerfield river hydroelectric development. (Power), Feb. 25, 1913. D. I.—Generation and transmission of hydroelectric power. E. A. Lof. (13), Jan., 1913. I.—Hydroelectric development on the Magpie River. (3), Jan. 23, 1913. I.—Hydroelectric developement replacing old power system. (15), Feb. 22, 1913. D. I.—Hydroelectric plant at Trollhattan, Sweden. (48), Jan. 11, 1913. D. I.—Hydroelectric power plant in San Juan, Argentina. (11), March 21, 1913.—Mill Creek automatic pumping station. (49), March 15, 1913. D. I.—Mount Hood hydroelectric development. W. P. Brereton and R. H. Mulock. (49), March 22, 1913. I.—Power development on the Deerfield River, Massachusetts. (15), Feb. 8, 1913. D. I.—Power developments on the Deerfield river, Massachusetts. (14), Feb. 15, 1913.—Prarie Du Sac hydroelectric plant. (39), April 1, 1913. I.—Water power from the Mississippi. G. F. Dell. (3), Jan. 29, 1913. D. I.

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Porto di Mannheim. D. Tramontani. (Annali della societa degli architett, Italiani), Jan. 16, 1913. D.

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Illinois waterway: The projects for a barge canal from Chicago to the Mississippi river. (12), March 5, 1913.

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Arrowrock dam, Boise irrigation project. (15), Feb. 22, 1913. D.—Concrete lining of the main canal on the Boise irrigation project. F. W. Hanna. (15), Dec. 21, 1912. I.—Economic aspect of seepage and other losses in irrigation systems. L. J. LeConte and W. C. Hammatt. (21), Dec., 1912.—Economic aspect of seepage and other losses in irrigation systems. L. Wagoner. (21), Jan., 1913.—Egyptian irrigation and the Assouan dam. (11), Dec. 20, 1912. D. I.—General method of constructing a rock fill dam 125 feet high. (12), Dec. 25, 1912. D. I.—Irrigation and river control in the Colorado River delta. L. J. LeConte and M. Knowles. (21), Feb., 1913.—Irrigation in the Sudan. P. F. Martin. (10), Jan. 31 and Feb. 14, 1913. D.—Irrigation project on the Price River in Eastern Utah. (15), Dec. 28, 1912. I.—Meeting of irrigation engineers at Fort Collins. S. H. Lea. (14), Dec. 26, 1912.—Problem of interstate swamp drainage. The Dismal Swamp. M. O. Leighton. (12), March 12, 1913. D.—Reservoir and canal losses in irrigation. (14), March 27, 1913. D. I.—Twin Falls: Oakley irrigation project. A. M. Korsmo. (14), Dec. 13, 1912. D. I.—Wapato irrigation project in Nothern Washington. (15), Jan. 4, 1913. D. I.—Wellwater irrigation system in Southern California. (15), March 1, 1913. D. I.—Yakima Indian reservation drainage project. J. W. Martin. (14), Feb. 20, 1913. D. I.

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Are levees a failure? J. W. Price. (27), March 1, 1913.—Sacramento River flood control. F. H. Tibbitts. (12), April 9 1913. D. I.—Various methods of constructing levees. (39), Jan. 15, 1913. I.

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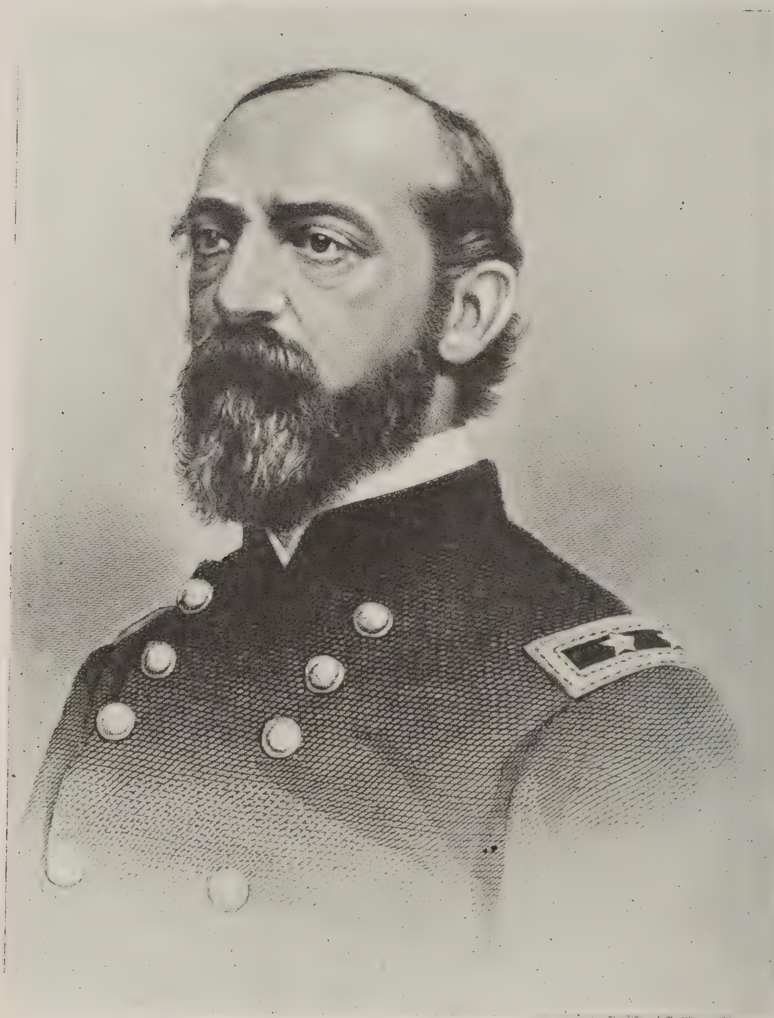


GEN. GEORGE GORDON MEADE

CORPS OF ENGINEERS, UNITED STATES ARMY

1842-1872

BORN 1815—DIED 1872

SEE P. 493

The Dalles-Celilo Canal

BY

MR. FREDERICK C. SCHUBERT
Member American Society of Civil Engineers

The object of the Dalles-Celilo Canal is to overcome the obstructions in the Columbia River between the foot of Three Mile Rapids, 200 miles from the Pacific Ocean, and the head of Celilo Falls, 9 miles distant. These obstructions constitute the only barrier remaining, after the construction of the Cascades Canal (45 miles below) to continuous navigation of the Columbia River from the Pacific Ocean to Priest Rapids, Washington, a distance of 407 miles.

COLUMBIA RIVER.

The Columbia River rises in the southwestern part of British Columbia, flows northwesterly through that province about 200 miles, thence southerly through British Columbia and the State of Washington about 700 miles to the Snake River, and finally westerly for about 300 miles between the States of Oregon and Washington into the Pacific Ocean. It is the third largest river in the United States, having a total length of approximately 1,200 miles, 750 of which are in the United States, with a drainage area of 259,000 square miles. Its low water flow is approximately 50,000 second-feet, while its high water flow is estimated at 1,500,000 second-feet. The total navigable waters of the river and its tributaries aggregate 2,136 miles.

The Columbia River is navigable for ocean-going vessels from the Pacific Ocean to Vancouver, Wash., a distance of 106 miles. From Vancouver to the foot of the Dalles or Five Mile Rapids, a distance of 94 miles, including the Cascades Canal, there is a low water channel depth of 8 feet. Between the foot of Dalles Rapids and the head of Celilo Falls, the river is not navigable on account of swift currents and the falls at Celilo. From Celilo to Priest Rapids, a distance of 198 miles, there is a least-channel depth, at low water, of 4½ feet. At Priest Rapids the river is obstructed

for 11½ miles. From the head of Priest Rapids to Arrow Head Landing, B. C., a distance of 488 miles, there are long stretches of navigable water broken by obstructions. In this latter distance there are 255 miles of navigable waters, 113 miles of river navigable at favorable stages, 109 miles (Okanagon to Spokane Rapids) navigable at great risk and only 11 miles (Rickeys Landing to Marcus) considered not navigable.

On the Snake River there is a minimum channel depth of 30 inches at low water, from its mouth in the Columbia River to the Grande Ronde River, a distance of 165 miles.

OBSTRUCTIONS OVERCOME BY THE DALLES-CELILO CANAL.

There are four principal obstructions to navigation in the stretch of river between the foot of Three Mile Rapids and the head of Celilo Falls, where at each place a rock reef crosses the river bed and through which the river has cut its way. The lower three of these obstructions take their names from their approximate distances above the town of "The Dalles." The fourth, or upper reef, is called Celilo (or Tumwater) Falls.

At Three Mile Rapids the general surface of the reef is from 65 to 75 feet above sea level and about 30 feet above low water in the river. At the head of these rapids the river has worn a channel through the solid rock to a depth of 165 feet below low water, making a total depth from the general surface of the reef to the bed of the river of about 200 feet. While the natural width was about 190 feet at the narrowest place and quite crooked, it has been so improved by the removal of rock reefs that it is now generally 250 feet in width with a least depth of 10 feet at low water. This channel can now be navigated at any stage of the river by steamboats of about 600 horsepower, which is about the usual power of boats operating on the upper river. The most difficult stage for navigation is about 15 feet above low water, at which time the entire flow of the river must pass through this narrow channel. After the 15-foot stage is passed the flood waters pass through high water channels.

The second obstruction to navigation is Five Mile Rapids (the Dalles of the Columbia) where although the general surface of the reef (1½ miles in length) is from 130 to 150 feet above sea level, the low water surface is only 55 feet above sea level. The channel cut by the river has a depth of 160 feet at low water at the head of the rapids and a width, at its narrowest point, of 160 feet. Until a 35-foot stage of the river is reached at the head of

U S Engineer Office, Portland, Ore.,
April 20, 1908.
my report of this date to the
eers, U.S.A.
J. W. Roessler
Lieut-Col, Corps of Engineer


VICINITY MAP
OF THE
DALLES-CELILO CANAL
Project of 1908
Scale 1/12000
S.W. ROESSLER, Lieut.-Col. Corps of Engineers, U.S.A.
J. S. POLHEMUS, Asst Engineer
F. C. SCHUBERT " "
CHAS. E. BARTEAU, Del.
U.S. Engineer Office, Portland, Ore.
April 20, 1908.
To accompany report of this date to the
Chief of Engineers, U.S.A.
S. W. Roessler
Lieut. Col. Corps of Engineers, U.S.A.
THE DALLES
Columbia River
Big Eddy
SEUFERTS
Five Mile Rapids
CAPE HORN
CELILO
N
W
E

D-196

the rapids, the entire flow of the Columbia River must pass through this gorge. At the head of the rapids the ordinary stages reach a height of 75 feet, while the maximum flood (1894) reached a height of 93 feet above low water. At low water the fall of the rapids through the gorge, 1½ miles in length, is about 10 feet, and at ordinary floods (75-foot stage) about 40 feet. The rapids are not navigable at any stage owing to the swift currents and dangerous rocks. Several steamboats have run the rapids at a moderately low stage of the river, but not without considerable damage to the hulls.

The third obstruction is Ten Mile Rapids. At this point a rock reef extends about half way across the river bed. The channel at the head of the rapids at low water and up to a stage of 17 feet higher is a gorge through the solid rock about 250 feet in width. After a rise of 17 feet the flood water passes through a large high water channel along the right bank. The fall through the rapids at low water is about 2½ feet and at high water about 5 feet. The rapids are not particularly dangerous to navigation, as the channel is straight and deep. The rise of the river at the head of the rapids during ordinary June freshets is about 70 feet, with a maximum flood stage of about 80 feet.

The fourth obstruction is Celilo or Tumwater Falls, where a rock reef crosses the river bed. This reef has a surface elevation of about 132 feet above sea level. The low water surface above the falls is 126½ feet above sea level and at the lower end of the reef 80 feet, making a total fall of 46½ feet at low water. The lower end of the reef is 6,000 feet below the head of the falls. At low water the river bed is composed of a large number of small islands separated by deep channels. The main channel is about 300 feet in width and follows the Oregon shore. At the head of the main channel is the noted Horse Shoe Falls, where there is a sheer fall of about 24 feet at low water. At ordinary high water during June freshets, the fall is about 6 feet. During the June freshets a boat can run the rapids without great danger, and several steamboats have done so without serious damage to the hulls.

At extreme low water the total fall in the river between the head of Celilo Falls and the foot of The Dalles or Five Mile Rapids is 81½ feet, and at high water, during ordinary June freshets, about 50 feet. During low water the fall is mostly concentrated at Celilo Falls, and at high water at the Dalles Rapids. On account of the flattening of the river slope at the rapids during the freshet

season, the extensive development of water power at the rapids is a difficult matter, and only a small part of the total flow of the river (about 50,000 second-feet at low water and 1,500,000 second-feet at high flood stage) can be utilized. To utilize the total fall of the river in the above-mentioned stretch would require a ditch or flume 8 2-3 miles in length, which would be a difficult undertaking on account of a railroad on the north bank and the Celilo Canal and a railroad on the south bank. A limited amount of water power can be obtained from water from the canal for use in operating lock gates, bridges, and for electric lighting.

NAVIGATION OF THE COLUMBIA RIVER.

From the date of its discovery, May 11, 1792, to 1850, the navigation of the Columbia was confined to the river below the Cascades and principally between Portland and the sea. Sea-going sailing vessels were in use until the *Beaver*, the first steam vessel, entered the Columbia River in March, 1836.

The first river steamer to ply the river was the *Columbia*, a side-wheel steamer 90 feet in length and 16-foot beam, which made her trial trip July 13, 1850. The *Lot Whitcomb*, a side-wheel steamer 160 feet in length and 24-foot beam, was launched December 25, 1850.

The first steamer to ply the river between the Cascades and The Dalles was the *Jason P. Flint*, a small iron propeller boat built below the Cascades in 1851 and hauled up over the Cascades. The *Flint* returned to the lower river in 1852.

The steamers *Mary* and *Hassalo* were built at the Cascades in the years 1853 and 1857, respectively, and the steamer *Idaho* at the upper Cascades in 1860. All three plied between the Cascades and The Dalles.

The first steamer to ply the river above Celilo Falls was the *Col. Wright*, built at Celilo in 1858, which made her first trip in April, 1859. During the years 1859-1860, the portage at the Cascades was made on wooden tramways and from the Dalles to Celilo by teams. Later, portage railroads were constructed at the Cascades and from The Dalles to Celilo.

The first steamer to run on the Snake River was the *Col. Wright*, which made a trip in June, 1860, up the Snake to Clearwater River and 37 miles up the Clearwater. The *Okanogan* followed a few weeks later.

Lewiston, Idaho, at the confluence of the Snake and Clearwater rivers, was founded in 1860. In 1862, the rush to the Salmon

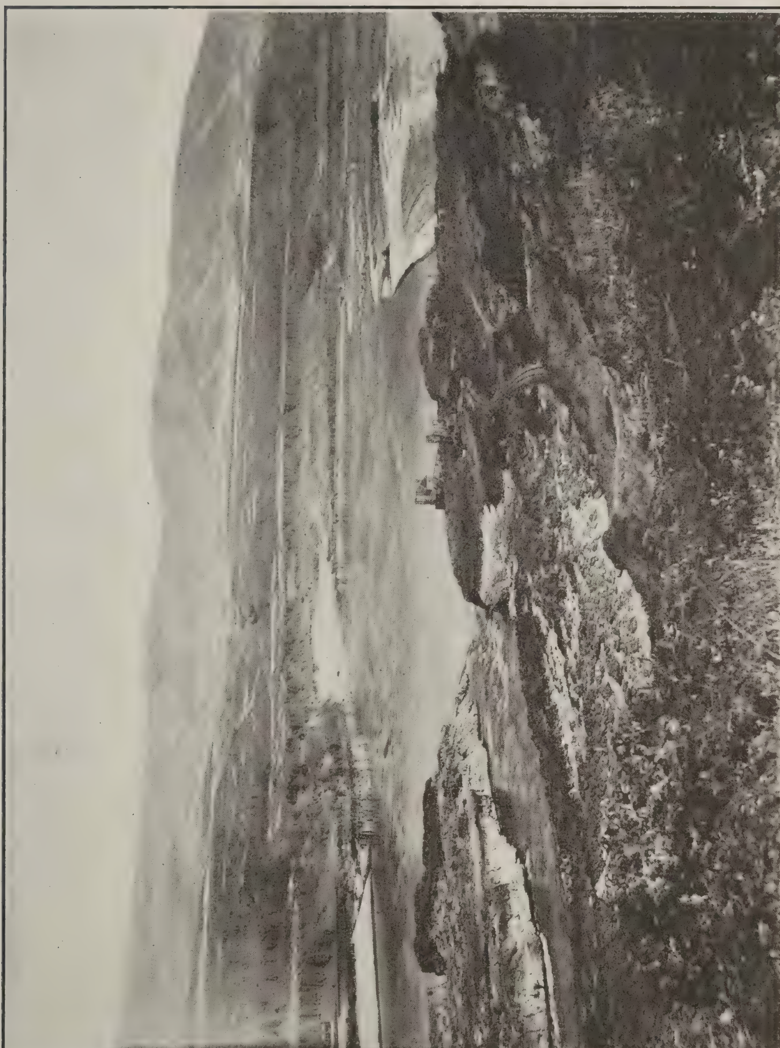


Fig. 1. Head of Five Mile Rapids, looking upstream about northeast, from Oregon shore.

River mines in Idaho required greater transportation facilities than existed at the time, and four river steamers were built. The *Okanogan, Tenino, Spray* and *Cascadilla.* The *Okanogan* had a length of 118 feet, 24-foot beam and 5½-foot depth of hold. The *Tenino,* 136 feet in length, 26-foot beam and 5-foot depth of hold. The *Cascadilla* and *Spray* were smaller boats.

The business during the year was enormous, hardly a steamer carried less than 200 passengers and freight was refused. On April 29, 1862, the steamer *Tenino* left Celilo for Lewiston with a load of passengers whose fares amounted to $10,954.00.

In 1863, the steamers *Webfoot* and *Nez Perce Chief* and a small propeller boat, *Celilo,* were added to the fleet above Celilo. All three boats were built at Celilo. The *Nez Perce Chief* was 126 feet long and 25-foot beam. The *Webfoot* was 160 feet long and 31-foot beam.

On October 29, 1863, the *Nez Perce Chief,* launched that year at Celilo, brought down gold dust valued at $382,000.00 from the Salmon River mines.

In the year 1864, the steamers *Yakima* and *Owyhee* were added to the upper river fleet, both boats being built at Celilo. The *Yakima* was 150 feet long and 29-foot beam. The *Owyhee* 123 feet long and 24-foot beam. During this year the transportation rates were as follows:

Portland to The Dalles, about 110 miles,	freight,	$15 ton;	fare, $6
Portland to Umatilla, about 200 miles,	freight,	45 ton;	fare 10
Portland to Walla Walla, about 250 miles,	freight,	50 ton;	fare, 12
Portland to Lewiston, about 360 miles,	freight,	90 ton;	fare, 22

In the year 1864, the boats between The Dalles and Lewiston carried 36,000 passengers and 21,834 tons of freight. The trip from Portland to Lewiston was made in from three to four days, and the down trips in two to three days. In 1911, the trip from Lewiston to Portland was made in less than one day with portage from Celilo to The Dalles (12 miles).

In November, 1865, the steamer *Forty-Nine* was launched at Colville, Wash., and in December, 1865, ascended the river to the head of Lower Arrow Lake and meeting floating ice, returned.

In 1866, the steamer *Mary Moody* was built on Lake Pend O'Reille, the first steamer on any of the lakes, except The Arrow Lake. The steamer *Shoshone* was built at Fort Boise during the year for navigation of the Snake River above the Seven Devils. This steamer made a perilous but successful trip to the lower Columbia River, but not without serious damage to its hull.

Navigation of the upper Columbia River declined rapidly from 1866 to 1870. In June, 1866, the steamer *Okanogan* was taken over the Celilo Falls to the middle river, and in June, 1870, the *Shoshone* and *Nez Perce Chief* followed.

During the 70's a number of the older boats were rebuilt and renamed. The *Yakima* became the *Annie Faxon* and the *Col. Wright* the *Spokane* in 1877. The steamer *Northwest* was also built in 1877.

In 1873, the Government commenced the work of improving the upper river above Celilo, by removing dangerous rocks, and in 1878 began the construction of the Cascades Canal on the middle river.

In 1878, the steamer *John Gates,* 151 feet long and 32-foot beam, was built at Celilo. In 1887, this steamer made a trip up the Columbia, passing over Priest and Cabinet rapids and part way up Rock Island Rapids, and then returned down river.

In 1880, the steamer *Fred K. Billings*, a steamer 200 feet in length, 37-foot beam and 6-foot depth of hold, was built at Celilo. She was probably the largest steamer ever operated on the upper Columbia, and was wrecked in 1894.

In February, 1881, the *Harvest Queen* was launched at Celilo and one week later was taken over Celilo Falls. The steamer was 200 feet long by 37-foot beam and 7½-foot depth of hold. It was probably too large to operate on the upper river and it seems strange that a steamer of such depth of hold should have been built for the purpose.

In 1882, the steamer *Katie Hallet* was built on the Clarks Fork of the Columbia for use in the construction of the Northern Pacific Railway. During the years 1883 to 1885, a number of steamers were built on the upper Columbia River near the lakes in British Columbia. On Lake Kamloops four or five steamers were built, all 125 to 140 feet in length and 25 to 28-foot beam. In 1886, several small steamers appeared on Lake Kootenia, British Columbia, and in 1890 another steamer was added to the fleet on the lake. Three steamers were also built for the Columbia River and Arrow Lake from Revelstoke to Little Dalles.

After the completion of the Oregon Railroad and Navigation Company Railroad from Wallula to Portland in 1882, navigation of the upper Columbia practically ceased. Only a few steamers remained on the river above Celilo Falls and these were taken to the Snake River to operate between Riparia and Lewiston, where there was no railroad.

During the years 1891 to 1894, an attempt was made to revive the navigation of the river when a company built a portage railroad on the Washington shore between the foot of the Dalles Rapids and Columbus. The steamer *Fred K. Billings,* rebuilt at Pasco, Wash., to run in connection with the *Portage,* was wrecked in 1894 on its first trip down river. Shortly afterwards the company went into the hands of a receiver and nothing further toward navigation of the upper river was done for some years.

In 1896 the Cascades Canal was opened to navigation, which gave continuous water navigation from the Pacific Ocean to the foot of The Dalles or Five Mile Rapids. On February 19, 1903, the State of Oregon appropriated $165,000.00 for the construction of a portage railroad around the obstructions between the foot of The Dalles Rapids and the head of Celilo Falls, to be operated during the construction of the Dalles-Celilo Canal. This railroad was completed during the fall of 1905, and in 1910 was extended to The Dalles, Oregon.

During the year 1905 the steamer *Jerome,* built near Pasco, Wash., was used on the upper river above Pasco, Wash., and was intended for use in conjunction with the State portage at Celilo. The captain of the *Jerome,* who had never made a trip below the mouth of the Snake River, wrecked his steamer at Homily Rapids in September, 1905, while returning upstream to Kennewick on his first trip to Wallula Junction, after having abandoned the trip to Celilo.

In 1908, the railroad (J. J. Hill's road) along the north bank of the Columbia River was completed, giving railroad competition on both sides of the river. Two small steamers were in use on the upper river in connection with this portage road until 1908, when the Open River Transportation Company built two larger steamboats. The smaller boats have been taken off the river and there are now only two steamers running regularly between Celilo and Priest Rapids and to Lewiston, Idaho, when the Snake River stage is favorable. On the Snake River there are now two steamboats in use for connection with the railroads between Riparia and Lewiston.

EARLY EXAMINATIONS FOR IMPROVEMENTS.

There has been some criticism in regard to the adoption of a continuous canal for overcoming the obstructions in the river between the foot of the Dalles Rapids and the head of Celilo Falls, but the following brief history of the earlier examinations and

Fig. 2. General view of completed canal work at Celilo, showing the lock and boat basin, together with the main portion of the falls at low water.

prospects will show that every phase of the situation has been considered, that most of the earlier projects were more or less temporary in character and inadequate, and that a continuous canal is the best and cheapest method for overcoming the obstructions, and will meet all the requirements of navigation for many years.

The earliest examination on record of this locality, was made in 1874, under the direction of M. Michler, then Major, Corps of Engineers, U. S. A. It was then thought that Five Mile Rapids (The Dalles Rapids) could be made navigable for all stages of the river by the removal of the most dangerous rocks. Ten Mile Rapids was thought to be navigable at all stages, and that Celilo Falls could be passed by a canal and two locks. The north shore of the river was considered the better side for the canal and locks.

The Act of Congress of March 3, 1879, required a survey with plans and specifications for a canal and locks from Celilo to the foot of The Dalles Rapids. Surveys were made in the winter of 1879-1880 and the following plans were considered:

1. To carry the Celilo level by a canal, with a lock at upper end, to a flight of locks located near Big Eddy.
2. With lift locks, as in plan 1, but to dam the river, making the overfall into Big Eddy.
3. Open river improvement for all stages to Celilo Falls, and lockage for same at the falls.
4. Open river improvement for lower stages to the foot of Celilo Falls during low and mean high water at the falls; open passage over the falls for higher stages; and lockage from medium to the highest stage at The Dalles Rapids.

Plan 3, at an estimated cost of $7,674,495.51 was recommended. Nothing further was done in regard to the matter until 1892.

The provisions of the River and Harbor Act of July 13, 1892, required a reexamination of the obstructions to navigation in the Columbia River from below Three Mile Rapids to the head of Celilo Falls, and a Board of Engineers appointed for this purpose reported on same April 12, 1893.

They considered the following methods for overcoming the obstructions:

1. By portage railroad from Celilo to below Three Mile Rapids.
2. By a boat railway, the boats to be lifted by hydraulic power from the water on a car with wheels, which, upon arriving at the proper elevation, would be hauled by locomotives upon a railway

track to the other terminus and lowered by hydraulic power to the water below.

3. By a canal. From previous data relating to a partial canal, the Board had concluded that a continuous canal, on account of its great cost, was inadmissible, and the funds available were expended on surveys and plans for a portage railway. An estimate was made, however, from the data at hand, which was favorable enough to modify the opinions of the Board. The proposed canal was located on the Oregon shore, was designed to have a bottom width of 100 feet and a depth of 6 feet. The lock at Celilo was to have a lift of 20 feet at low water, thus raising the boat above ordinary floods. At Big Eddy, a height of 72 feet at low water was to be overcome by means of a hydraulic lift carrying a caisson in which the boat remains afloat. Intermediate between Celilo and Big Eddy there were to be two locks with lifts of 15 feet each. The water supply for the canal was to be by a feeder, 13,000 feet in length, from the Des Chutes River.

4. By a dam at the head of Five Mile or Dalles Rapids, to pond the water back to the foot of Celilo Falls.

This plan was favorably considered by the Board. It would have required a short canal with locks at Celilo, and another canal with locks from the head of Five Mile Rapids to Big Eddy. As no data were available at that time, the Board made no recommendation but suggested that the necessary surveys be made so that the plan could later receive consideration as an alternative proposition.

The recommendations of the Board of Engineers were for the construction of a standard gauge portage railroad from Celilo to The Dalles City, at an estimated cost of $454,390.00, and when the necessity should arise for greater accommodation than could be supplied by a portage railway, the construction of a canal located on the Oregon shore.

In a letter dated May 29, 1893, Col. G. H. Mendell, Corps of Engineers, U. S. A., recommended the construction of a boat railway from Celilo to Big Eddy. He also recommended the portage railway favored by the Board of Engineers' report, but to be so aligned and constructed that for most of its length it would form a part of the boat railway to be constructed later.

The River and Harbor Act of August, 1894, appropriated $100,000 for a boat railway from the foot of The Dalles Rapids to the head of Celilo Falls, at an estimated cost of $2,264,467.00, including open river improvement of Three Mile Rapids.

Up to June 30, 1899, no actual construction work for a boat railway had been done, but most of the right of way had been secured. Capt. W. W. Harts, Corps of Engineers, then reported that the navigation interests preferred a portage railway or a canal and locks. As no appropriation had been made since 1896, the Secretary of War decided to defer further action in the matter until Congress definitely decided the question of constructing a boat railway.

The River and Harbor Act of June, 1900, called for an examination of the river between the foot of The Dalles Rapids and the head of Celilo Falls, with a view to the construction of a canal and locks to overcome the obstructions to navigation.

Captain Harts then proposed a plan calling for a canal and locks around Celilo Falls, open river improvement from the falls to the head of Five Mile Rapids, a submerged dam at the latter place designed to raise the low water surface 20 feet and drown out Ten Mile Rapids, a canal and locks from the submerged dam to the foot of Five Mile Rapids, and open river improvement of Three Mile Rapids.

The proposed canals were to be 65 feet wide on the bottom with locks 300 feet in length and 54 feet wide at the gates, and providing for 7 feet of water over the miter sills. The total estimated cost of the above plan was $3,969,371.00. The Act of Congress dated June 13, 1902, approved the project of improving the river from the foot of The Dalles Rapids to the head of Celilo Falls in accordance with Captain Hart's plan, but provided that before entering upon the work an examination should be made by a Board of Engineers with a view to modifying the project so as to diminish the cost of the improvement.

Detailed surveys made in 1903, under the direction of a Board of Engineers, resulted in the recommendation of a continuous canal with four locks, located on the Oregon shore and extending from the head of Celilo Falls to the foot of The Dalles Rapids. The canal was to be 65 feet wide on the bottom with side slopes of 4 on 1 in rock sections, and a low water depth of 8 feet, with gravity walls and concrete lining in pervious sections. The locks were to be 40 feet in width and 250 feet in length between hollow quoins, with a depth of 7 feet on sills. There was to be one lock at Celilo, one at The Dalles Rapids, and a double lock at Big Eddy (foot of The Dalles Rapids). The total cost of the improvement, including open river improvement of Three Mile Rapids, was estimated at $4,121,331.46.

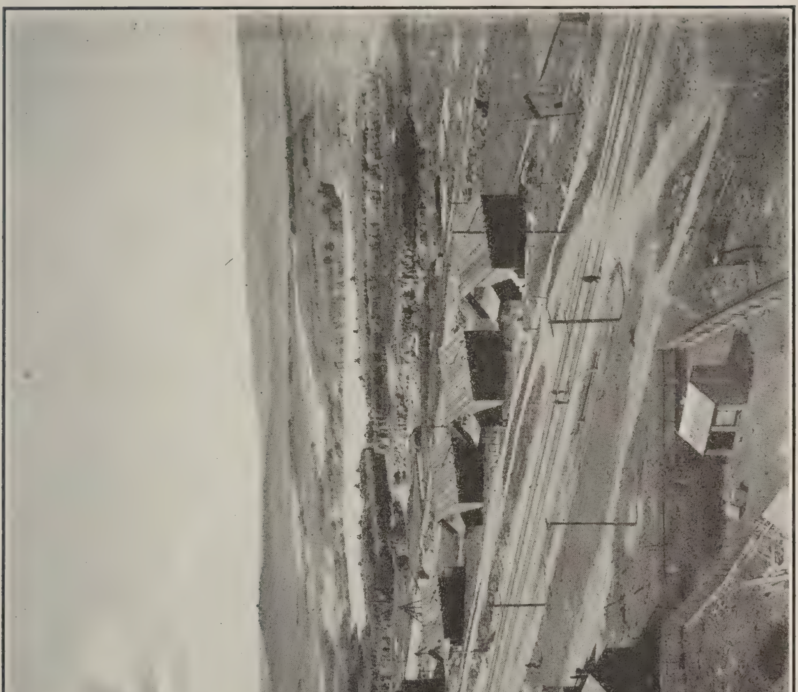


Fig. 3. Government construction camp No. 1, Big Eddy, Oregon, looking downstream about due west. Note Mount Hood under the clouds. The line of the canal is just beyond the houses. Big Eddy is body of water in right center of picture, with Five Mile Rapids opening into it about 1 inch from right edge of picture.

The above plan was approved by the Secretary of War November 6, 1903, with the proviso that no work should be begun until the right of way and release from damages were conveyed to the United States free of cost.

The Oregon State Legislature appropriated $100,000 for the purchase of the necessary right of way and in April, 1905, deeded the same to the United States.

The first contract for the project was let in 1904, and provided for the open river work at Three Mile Rapids. The second contract was for the construction of about ½ mile of the canal at Celilo, and was dated August 10, 1905. Actual construction on the canal began in October, 1905.

ALIGNMENT OF CANAL CENTER LINE.

The original center line of the canal upon which the Board's estimate was based, had a total length of 45,300 feet, of which 16,567 feet was on thirty-two curves.

A more careful study of the ground resulted in a relocation of the center line having a length of 44,357 feet, with 7,838 feet on fourteen curves. Of the fourteen curves, eleven occur in passing and boat basins in which the canal trunk is twice or more its normal width, so that practically all of the curvature is eliminated. Where the curves occur in the canal trunk the excavation has additional width to compensate for the curves.

The final location of the canal was considerably affected by the limits of the right of way which had been secured for the original alignment and by the existence of the O. R. & N. railroad grade on the south side of the canal line. The main obstacle to the canal relocation was the proviso in the Act of Congress that the right of way must be furnished without cost to the United States. The relocation required about 78 acres outside of the right of way acquired for the original location and to obtain the additional right of way it was necessary to have the owners of the adjacent lands agree to exchange some of their land for the canal right of way lands. The negotiations covered a period of five years, during which time one property holder along the canal line acquired all the lands necessary for the new location of canal center line for which he accepted an equal area of lands, acquired for boat railway and canal. The new right of way was thus acquired without cost to the United States other than that incurred for abstract of title, all other work being done as a part of the regular routine of the engineering force. It is one of the few instances where the

United States did not pay dearly for something it needed badly.

The following table shows the comparative curvature of the two alignments:

COMPARATIVE CURVATURE OF CENTER LINE.

Original Location.

Number of curves.	Degree.	Radius, feet.	Total angle.	Remarks.
1	1	5,730.0	2°44′	
3	2	2,865.0	47°30′	
3	3	1,910.0	36°34′	
9	4	1,432.5	180°42′	
14	6	955.0	442°32′	3 curves in boat basins.
2	8	716.3	64°32′	
32	--------	--------------	774°34′	

SUMMARY.

Total number of curves		32
Total curvature of center line		774°34′
Curvature in canal trunk	Feet	15,113
Curvature in boat basins	Feet	1,454
Total curvature, in feet		16,567

Final Location.

Number of curves.	Degree of curvature.	Radius, feet.	Total angle.	Remarks.
1	2	2,865.0	7°28′	In canal trunk.
10	3	1,910.0	179°01′	3 curves in boat basins at lock. 6 curves in passing basins. Canal trunk, where curves occur, has additional width to compensate for curvature.
1	4	1,432.0	23°03′	In canal trunk.
2	6	955.0	54°51′	1 curve in Celilo Basin; 1 curve west end Celilo Lock.
14	--------	-----------	264°23′	

SUMMARY.

Total number of curves		14
Total curvature		264°23′
Curvature in canal trunk	Feet	1,469
Curvature in boat basins	Feet	6,364
Total curvature	Feet	7,833

Total length of canal, 44,347 feet.

While the final location of the canal center line reduced its length by 943 feet, it did not really increase the total necessary excavation when it is considered that the present alignment has ten passing basins, including the large basins provided at the various locks

where the original plans provided for only six basins, located on tangents, with no provision for widening the canal excavation at the thirty-two curves. The straightening of the center line, while it threw the canal into deeper excavation, decreased the amount of masonry to a considerable extent, so that the total cost of the canal along the new alignment was actually less than along the original line and has the further advantage of being farther from the river and less exposed to the action of the freshets.

The following table gives the comparative quantities of excavations for the two alignments:

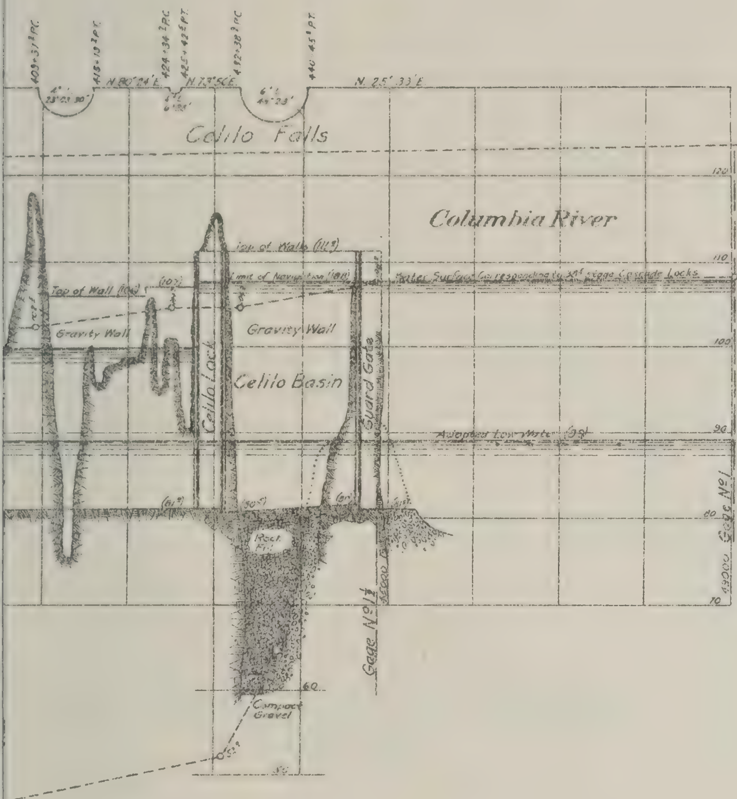
Material.	Original location. Estimate 1903.	Final location. Estimate 1908.
	Cubic yards.	*Cubic yards.*
Dry rock excavation	1,080,000	1,140,000
Sub-aqueous rock excavation	1,900	1,900
Gravel or sand excavation	840,000	998,000
Sub-aqueous gravel excavation	10,000	8,000
Rubble or concrete masonry	347,147*	143,600
Concrete, reinforced	------------	98,000

No reinforced concrete was specified for thin concrete linings in the original plan, while all such in the plans of 1908 are to be reinforced with steel rods. No land walls were estimated for in the original project but are included in the 1908 estimate for masonry. This estimate also includes masonry for an additional lock at Ten Mile Rapids.

CANAL TRUNK.

The original plan provided for a canal taking its water supply from the pool above Celilo Falls with a lock near the entrance, a pool 6½ miles long to Five Mile Rapids, a 10-foot drop, with a lock at this point, a second pool of about 1½ miles in length, and a drop of 70 feet into the Columbia River at Big Eddy (the foot of Dalles Rapids), with a tandem lock each with a lift of 35 feet. The low water level of the pool above Celilo Falls fixes the low water level of the Celilo Lock and the pool between Celilo and Five Mile Locks, the Celilo Lock being kept open until the river reaches a 5-foot stage, during which time all lockage is done at Five Mile and Tandem Locks. The height of the lock gates and lock walls are determined from the condition that the canal must pass boats until the river rises to a point where navigation at the Cascade Locks, 45 miles below, is suspended. The height of the

*Does not include sub-foundations for proposed gravity walls on sand and gravel foundations.



DALLES-CELILO CANAL

LONGITUDINAL SECTION

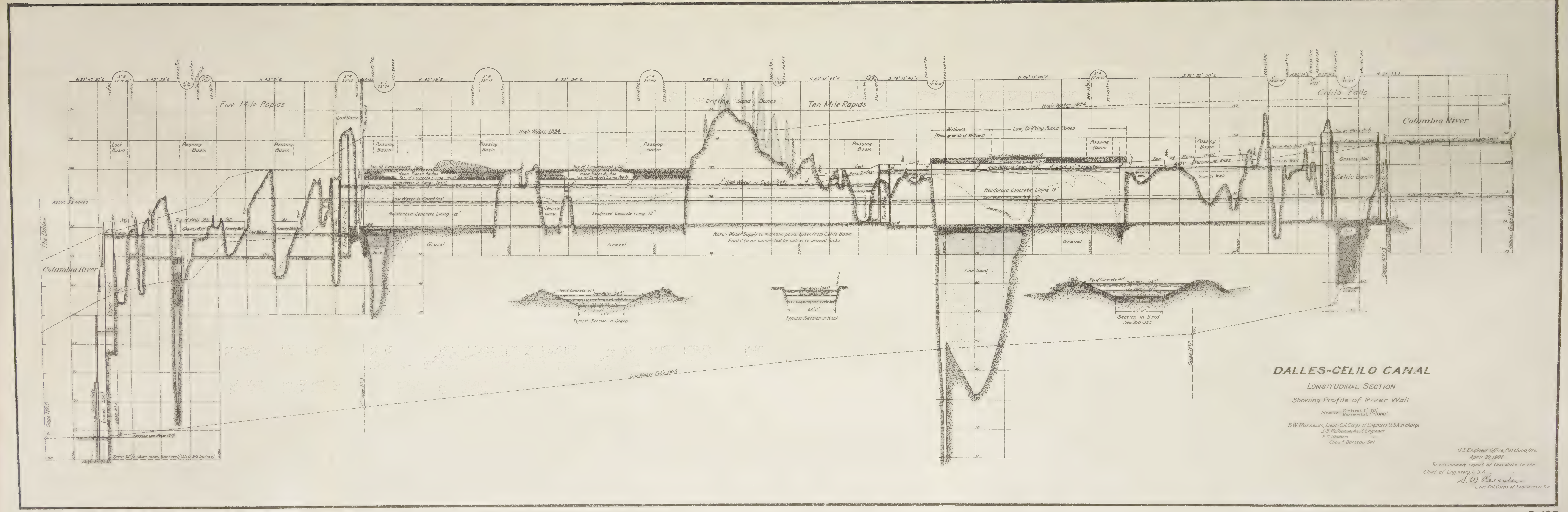
Showing Profile of River Wall

Scales { Vertical, 1" = 10'. Horizontal, 1" = 1000'.

S. W. Roessler, Lieut.-Col. Corps of Engineers, U.S.A. in charge
J. S. Polhemus, Ass't Engineer
F. C. Schubert, " "
Chas. E. Barteau, Del.

U.S. Engineer Office, Portland, Ore.,
April 20, 1908.
To accompany report of this date to the
Chief of Engineers, U.S.A.
S. W. Roessler
Lieut.-Col. Corps of Engineers U.S.A.

D-192



Five Mile Rapids
Ten Mile Rapids
Drifting Sand Dunes
Celilo Falls
Columbia River
High Water 1894
Lock Basin
Passing Basin
Reinforced Concrete Lining 12"
Gravel
Fine Sand
Willows (Thick growth of Willows)
Low, Drifting Sand Dunes
Celilo Basin
Gravity Wall
Columbia River
The Dalles
About 3.8 Miles
Note:- Water Supply to maintain pools taken from Celilo Basin. Pools to be connected by culverts around locks.
Typical Section in Gravel
Typical Section in Rock
Section in Sand
Low Water Feby 1905
DALLES-CELILO CANAL
LONGITUDINAL SECTION
Showing Profile of River Wall
Scales: Vertical, 1" - 10'. Horizontal, 1" - 1000'.
S.W. Roessler, Lieut-Col. Corps of Engineers, U.S.A. in charge
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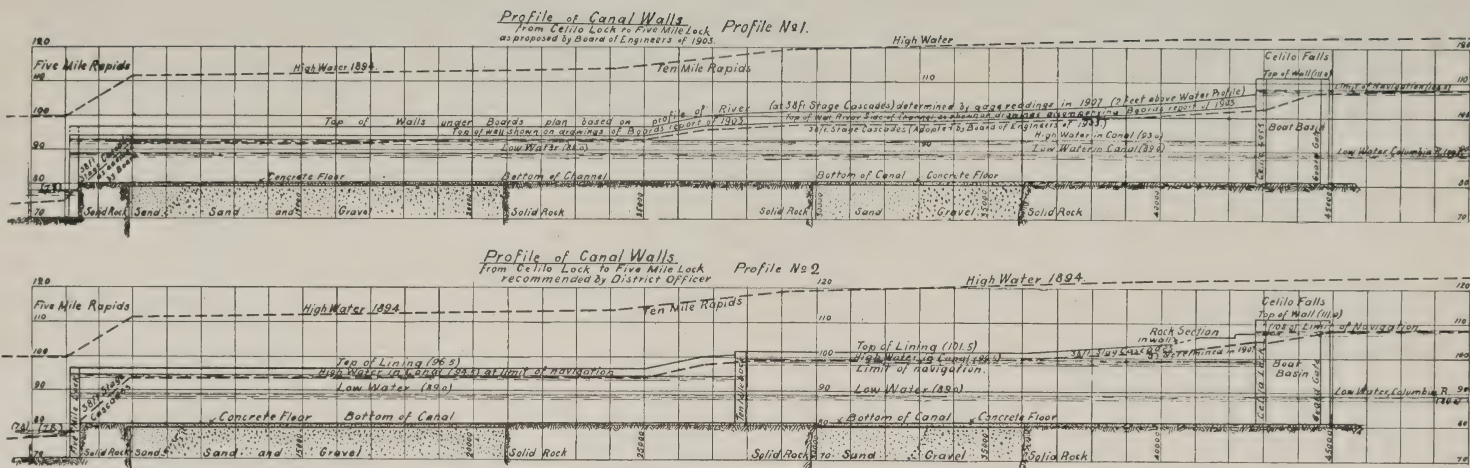


Fig. 4. Comparative profiles of canal walls, between Celilo Lock and Five Mile Lock. Upper profiles show water level and levels of tops of walls as proposed originally. The lower profile shows the locations of locks and levels of walls between locks as actually being constructed.

retaining walls and embankments of the canal trunk between locks is determined from the condition that they must exclude the river water until navigation is suspended at Cascade Locks. As a matter of fact, the river walls will have a slope parallel to but 2 feet higher than the river slope at the limiting stage.

When the survey for the original plans was made, the profile of the water surface from Celilo to Big Eddy at the limiting stage was determined during a falling river. However, upon further investigation it was found that the water surface between Celilo and the head of Ten Mile Rapids was about 2 feet higher during a rising river than during a falling river, with Celilo gauge readings remaining the same. After several years of gauge reading, a satisfactory profile was obtained and the tops of the walls made to conform with the same. In order to regulate the "drowning out" of the canal, and to avoid overfalls on the walls and swift currents in the canal, openings were left in the river walls, the bottoms of the openings being at the elevation of the surface of the river at the limiting stage. As soon as the river reaches the limiting stage it will begin to flow into the canal and fill the same before the river flows over the tops of the walls and embankments.

According to the original plans it was intended to plan Celilo Lock in solid rock close to the canal entrance at Celilo and to use the depression just west of the lock location for a boat basin, to be formed by means of a gravity wall on the river side and the natural rock surface on the land side. This wall was to be a gravity section about 53 feet in height, constructed of rubble masonry. It was found that the surface material on the site of the proposed wall was, for a greater part of the distance, boulders and cement gravel of such depth as to require deep excavation below low water or that means to cut off the flow of water under the wall would have to be provided. It was therefore decided to cross the depression, which was about 25 feet below canal bottom grade, by means of some type of aqueduct having an impervious lining.

After careful study it was decided to fill in the depression with rock, gravel and sand to the underside of a canal floor lining at grade, and form the river side with a wall having an embankment on its river side. The embankment has its toe over 100 feet from the foundation of the river wall, with a slope of 1 on 3, and heavily riprapped to prevent any possibility of the river's undermining the wall. As constructed, the wall is a gravity section from its top

to a point 3 feet below the top of the floor. Below this point the wall sets on a hollow block foundation, the blocks being filled with sand and gravel deposited in water to facilitate settlement.

While considering the above method for crossing the depression it was found that the floor of this aqueduct would be subjected to an upward water pressure due to a head of 12 feet when the water in the canal below Celilo Lock was at its highest level and to an upward pressure due to a head of 26 feet should the pool, below Celilo Lock, be emptied by accident. By locating the lock in rock at the downstream end of the depression, the water over the floor would at all times have the same elevation as the upper river and the upward pressure would be overcome. The lock was therefore located about ½ mile below the canal entrance and the river wall above the lock moved toward the river, thus forming a large boat basin which will accommodate several boats while awaiting lockage downstream. At the entrance to the canal a pair of guard gates with filling valves will shut out the river, while the lower end of the boat basin and the lock are connected to the river by a tunnel through solid rock. In this way repairs to the canal basin and lock can be easily made, during low water stages, by closing the guard gates and draining the basin and lock.

For 6,800 feet below Celilo Lock the canal is in solid rock excavation, the surface of which for the greater part of the distance is below the level of the limiting stage of high water navigation, the river water being excluded up to that stage by means of a concrete wall. As before stated, openings in the river wall, at convenient points, with their bottoms at the limiting stage of navigation, permit the river water to enter the canal and fill it before the river tops the wall. These openings will also regulate the extreme height of water in the canal.

Below the rock section, for a distance of 6,300 feet downstream, the canal passes through a gravel bar with a sand bar at its lower end. The depth of the sand is 80 feet below canal grade at its lower end. The original plan provided for gravity side walls with a floor of plain concrete 12 inches thick to form the canal section in sand and gravel. The floor to be protected against upward pressure by means of valves in the floor.

Upon investigation of the upward pressure in this stretch of canal, it was found that at the limit of navigation the river water would be from 6½ to 8 feet higher than the water in the canal pool below Celilo Lock, and that the concrete

floor would therefore be subjected to an upward pressure due to the difference in water levels. As the river rose and topped the walls, there would be an overfall into the canal unless provision was made to quickly fill the pool by means of connection with the river before the latter topped the walls. It was therefore decided to raise the water level of the pool in the canal between Celilo Lock and the head of Ten Mile Rapids, during the high water stages, by means of a lock located at the head of Ten Mile Rapids. This lock will begin service as soon as the river above Celilo Falls reaches a 5-foot stage and will keep the water in the pool below Celilo Lock at the same or higher elevation than the river water and as the river rises the canal water will rise with it, thus eliminating all upward pressure. When the two waters rise and meet at the weir levels, 2 feet below the tops of the walls, the canal will form a part of the river and be ''drowned out'' without danger to the walls or embankments.

The introduction of a lock at Ten Mile Rapids effected a great saving in the cost of overcoming the upward pressure on the canal floor, since making the floor strong enough to resist the upward pressure would have cost many times the cost of the lock, because the lock, on account of its location in solid rock, requires only two pairs of gates and a small amount of concrete work, part of the cost of which is compensated for by the less width of excavation required for a lock than for an equal length of canal. This lock will have a width of 50 feet to afford easy passage for boats when the lock is not in use.

The gravity walls formerly contemplated in sand and gravel sections would have required very expensive subfoundations on account of the depth of sand and gravel for a distance of 6,300 feet. It was therefore decided to form the canal prism by means of embankments above the natural ground surface and cover the waterway, up to 2 feet above the limit of navigation, with a reinforced concrete slope lining 12 inches in thickness. The canal prism in pervious ground to have side slopes of 1 on 2. There is now no danger of damage to the concrete lining by upward pressure. The cost of the concrete lined embankments is only a fraction of the cost of gravity walls and expensive foundations for same in sand of depths up to 82 feet. The cost of the embankments, except for the concrete lining, is negligible, being absorbed in the cost of hauling the excavated material from the excavations, more than one-half

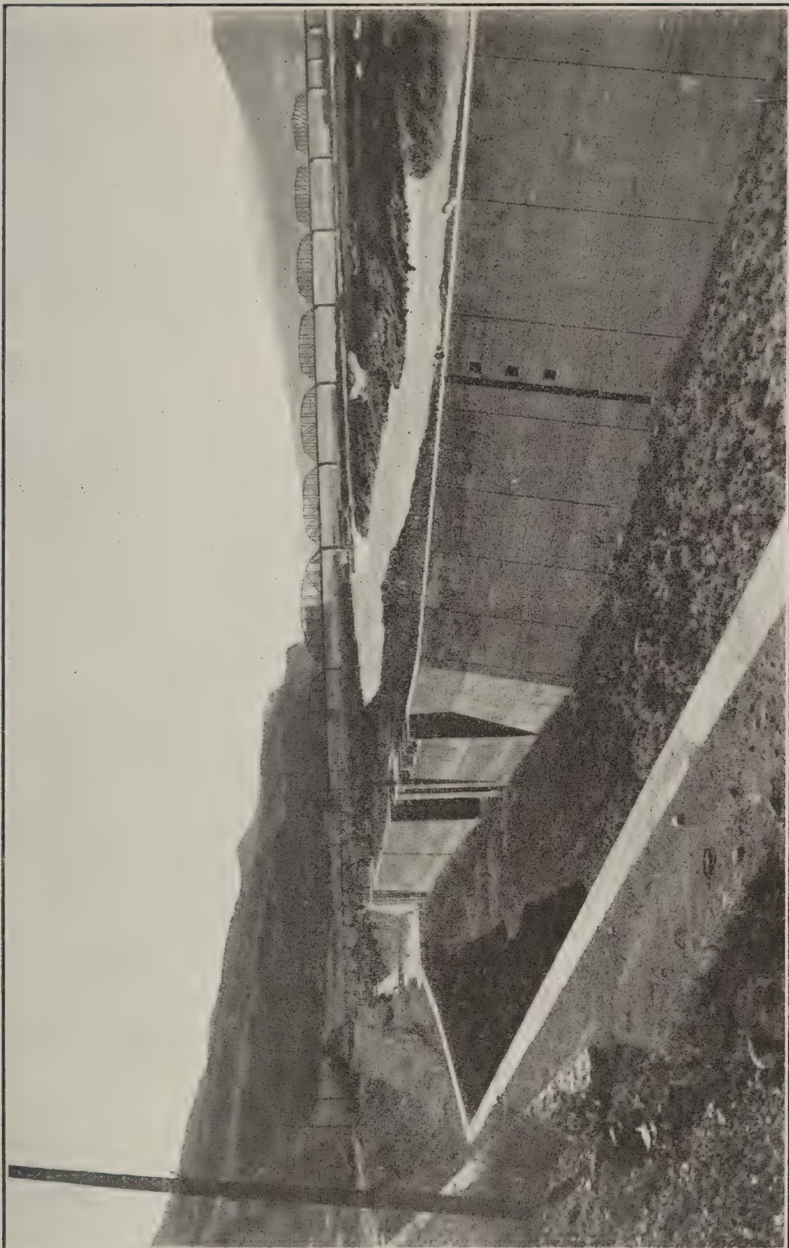


Fig. 5. Looking west from point above boat basin, Celilo, showing Celilo Lock and Oregon Trunk Railroad Bridge across the Columbia.

of which would probably have been hauled for back-filling gravity walls had the latter been built.

From the west end of Ten Mile Lock for a distance of 7,000 feet the canal prism is in solid rock excavation and presents no special features, but in this stretch of canal the rock surface, for a mile in length, and from the river to the bluffs, is covered with shifting sand dunes, the treatment of which will be mentioned later.

Where the surface of the rock is below the limiting stage of navigation, the river side of the canal will have a concrete wall, the top of which will be about 2 feet above the limiting stage of navigation and have openings to control the "drowning out" of the canal.

Below the above-mentioned rock section, for a distance of about 11,000 feet, the canal passes through gravel bars with rock reefs across the same, so that in places the canal section will be in rock. The ground surface for most of this distance is below the level of high water in the canal and embankments will be used to form the canal section above the natural surface of the ground. A reinforced concrete lining will be used where the canal is in sand and gravel.

To control the flood waters and regulate the water level in the pool below Ten Mile Lock, a number of openings are to be left in the river wall.

The sand bar ends at the head of Five Mile Rapids, where the canal enters solid rock and remains in rock excavation to the western terminus of the canal at Big Eddy, 9,000 feet distant.

The original plan had the location of the Five Mile Lock about 2,000 feet below the head of Five Mile Rapids. This location would have exposed the gates to an overfall of 14 feet during the high water stages, due to the river passing through the canal excavation at the head of Five Mile Rapids. The location of Five Mile Lock was therefore changed to the high rock reef at the head of the rapids, and the upper gate made high enough to exclude even the maximum flood of 1894. While the new location increased the amount of rock excavation on account of extending the lower pool eastward, the less amount of concrete necessary for the lock in its new location quite balances the cost. The new location of the lock, in addition to eliminating the danger to the lock from floods pouring over it, gave a longer pool between Five Mile Lock and Tandem Locks at Big Eddy, thereby providing a greater storage basin for water for lockage at Big Eddy. It was estimated that at

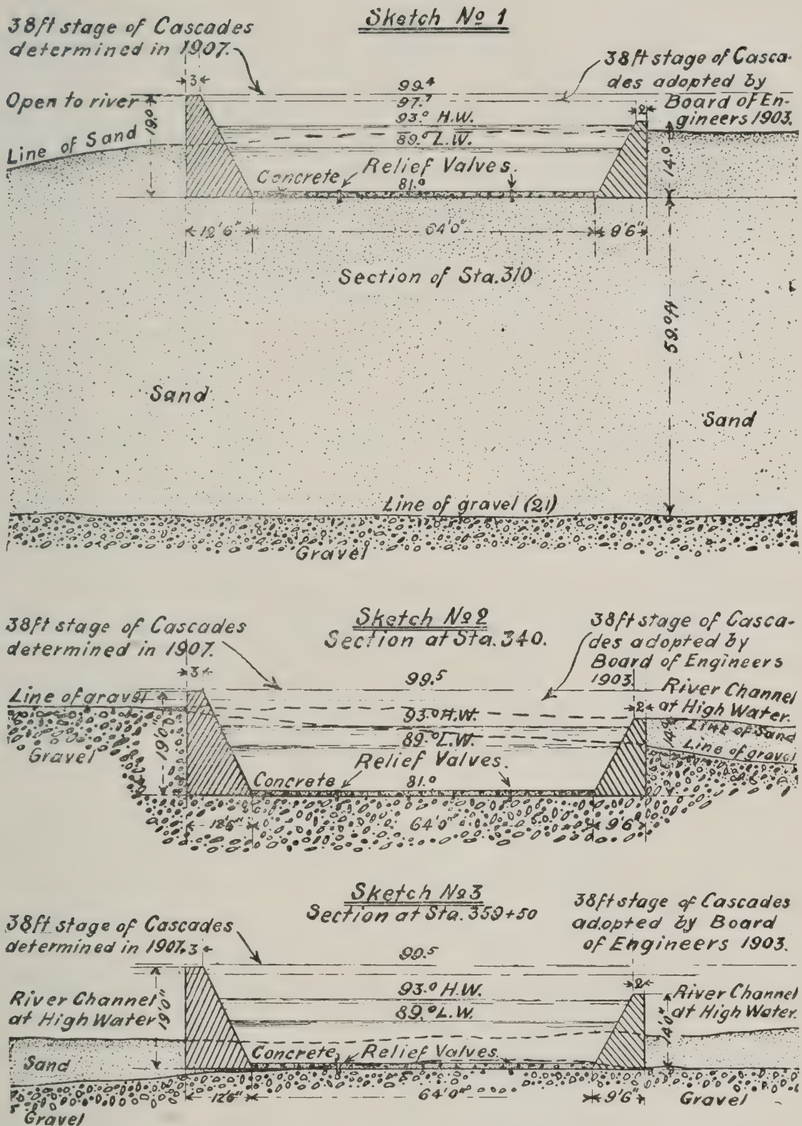


Fig. 6 Typical cross sections of canal walls and floors in sand and gravel, as adopted originally by Board of Engineers. Disposition of excavated material not shown.

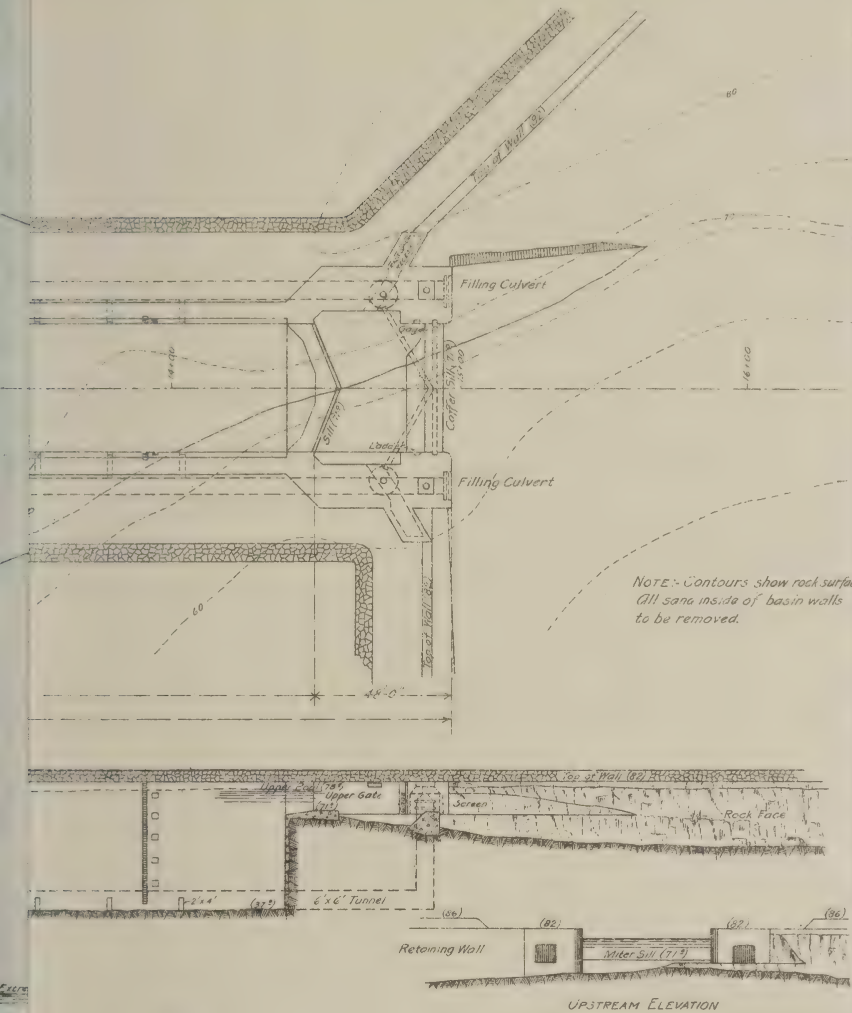
low water the passage of several boats at Big Eddy would lower this pool faster than it could be supplied from the pool above Five Mile Lock, and that boats in the pool would ground on account of lack of depth of water, but the increased length of pool and the several large boat basins placed in the pool, in the revised project, will provide sufficient storage, so that several boats can be locked through Tandem Locks in a short length of time without lowering the pool sufficiently to interfere with navigation. The flood water level in the pool between Five Mile Lock and Tandem Locks is regulated by means of openings in the river wall just as in the upper pools of the canal. The water level in the pools between the various locks will be maintained at all times, except during floods, by means of culverts around the locks.

LOCKS.

The original plans had locks having 40 feet width at the gates and 250 feet between hollow quoins. These dimensions were changed to a width of 45 feet and a length of 300 feet between hollow quoins to allow the passage of large-sized river steamboats. The length of the locks where there is a lift wall, as in the lower one of the tandem locks, is 310 feet between hollow quoins, to make all of the lock chambers of equal capacity. The depth of 7 feet on the sills was not changed.

The original plans contemplated an excavation in the solid rock large enough to permit the construction of gravity side walls, which contained, in the case of the tandem locks at Big Eddy, the filling culverts and in the case of the other locks, culverts used to maintain the water level in the pools below Celilo Lock. The plans were modified so that the lock chamber is formed by the excavation in solid rock, wide enough to allow for a concrete face wall to cover the rough rock surface. This face wall or lining has a minimum thickness of 18 inches and maximum thickness of 3 feet, and is to be securely anchored to the rock surface by means of steel rods let into the solid rock. The side walls of the lock chambers will have a batter of 24 on 1, and this, in addition to the support given by the rough surface of rock, will add to the stability of the wall. To avoid outward pressure from water which might get between the wall and the rock surface, the lining wall will be thoroughly drained by means of drain tile through the wall.

The filling culverts for the Tandem Locks at Big Eddy and the culverts around the locks at Celilo, Ten Mile Lock and Five Mile



DALLES-CELILO CANAL

PROPOSED DESIGN FOR

TANDEM LOCK AT BIG EDDY

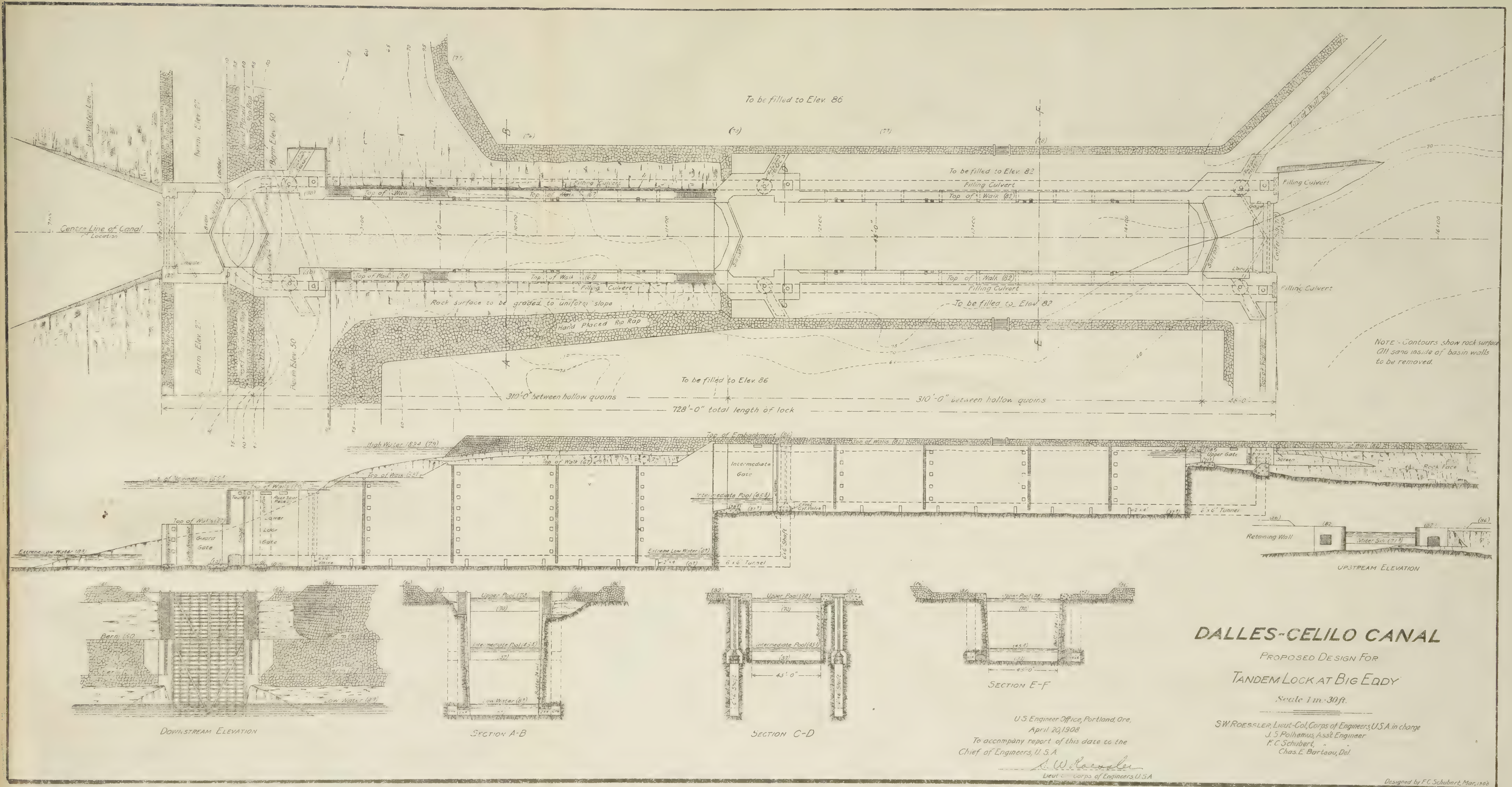
Scale 1 in.-30 ft.

S.W. ROESSLER, Lieut-Col, Corps of Engineers, U.S.A. in charge
J. S. Polhemus, Ass't Engineer
F. C. Schubert, " "
Chas. E. Barteau, Del.

...neer Office, Portland, Ore,
...' 20, 1908
...t of this date to the
...S.A.
...l W. Roessler
...ieut-Col, Corps of Engineers, U.S.A.

Designed by F. C. Schubert, Mar, 1908

D-193



DALLES-CELILO CANAL
PROPOSED DESIGN FOR
TANDEM LOCK AT BIG EDDY
Scale 1 in.=30 ft.
S.W. ROESSLER, Lieut-Col, Corps of Engineers, U.S.A. in charge
J. S. Polhemus, Asst. Engineer
F. C. Schubert, "
Chas. E. Bartaau, Del.
U.S. Engineer Office, Portland, Ore.
April 20, 1908.
To accompany report of this date to the
Chief of Engineers, U.S.A.
Lieut. Col. Corps of Engineers, U.S.A.
Designed by F.C. Schubert, Mar. 1908
To be filled to Elev. 86
To be filled to Elev. 82
Filling Culvert
Top of Walk (67)
Top of Walk (82)
Top of Walk (58)
Centre Line of Canal
Rock surface to be graded to uniform slope
Hand Placed Rip Rap
Berm Elev. 21
Berm Elev. 50
310'-0" between hollow quoins
728'-0" total length of lock
NOTE - Contours show rock surface. All sand inside of basin walls to be removed.
Top of Embankment (86)
High Water 1894 (79)
Intermediate Gate
Intermediate Pool (45.5)
Guard Gate
Lower Lock Gate
Upper Gate
Extreme Low Water (19)
6'x6' Tunnel
6'x6' Shaft
Rock Face
Retaining Wall
Miter Sill (71.3)
UPSTREAM ELEVATION
Upper Pool (78)
Intermediate Pool (45.5)
Low Water (8.9)
45'-0"
DOWNSTREAM ELEVATION
SECTION A-B
SECTION C-D
SECTION E-F

D-193

Lock, to maintain the water levels in the pools, will be formed by tunnels in solid rock parallel to the lock chambers. The tunnels on each side of the Tandem Locks at Big Eddy are to be 5 by 7 feet and are connected with the lock chambers by means of seven filling ports, each 3 feet in diameter. Where the rock is unsound, the excavations for the culverts will be made large enough to permit a concrete lining, but where sound rock is found no lining will be used, the loss of head due to friction being compensated for by the increased size of the culvert, thereby giving the same flow as a lined culvert of smaller dimensions. Celilo and Ten Mile Locks will be filled and emptied through the lock gates. Five Mile Lock will be filled by means of a culvert having six ports through the lift wall and controlled by two cylindrical valves, each 6 feet in diameter.

LOCK GATES.

The gates for all of the locks (a total of eleven pairs of gates, including two pairs of guard gates) will be of the horizontally framed girder, double leaf, mitering type, constructed of steel. A minimum thickness of ⅜-inch metal will be used to allow for deterioration and wear. The rise of sill will be one-fifth of the span.

The girders will have parallel flanges, as the length of the gates is relatively small (27 feet), and the economy in weight by reducing the width at the ends would be offset by the cost of so doing.

The depth of the gates (3 feet) was determined for the highest gates, and all of the gates were made the same depth to simplify the shop work.

From the top of the gate downward the girders were proportioned for a water pressure load equal to that due to a load of one-third of the total head on the gate until a point about one-third the height of the gates was reached, to provide for loads due to gate contact at the sill. Below this point the girders were proportioned according to their depth below high water surface, to be safe in case the support at the sill should fail. In proportioning the girders the support of the lower pool was neglected, for it is possible for this support to fail.

A fiber stress of 10,000 pounds per square inch for compression and 12,000 pounds per square inch for tension was used in the computations. The web plates have a minimum thickness of ⅜ inch and are stiffened where necessary. The bottom girder of all

gates is reduced in width (to allow the gate to overlap the miter sill) so that the weight of the gate is about equal to the upward water pressure on the bottom girder.

There are two vertical diaphragms in each leaf. They were not designed to carry any of the load, but to stiffen the whole structure.

The sheathing consists of flat plates with a minimum thickness of 3/8 inch. The sheathing has the same thickness throughout each leaf, it being considered an advantage to have a uniform thickness of sheathing to avoid fillers where such thickness changes. The thickness of the sheathing was determined for the lowest panels in a leaf and the spacing of the girders determined from the sheathing thickness.

The spacing of girders, especially in the top part of the gates, was kept the same as far as possible for all gates to avoid too many different sizes of plates and details.

The quoin and miter posts are formed by a vertical plate inclosing the ends of the girders, to which is bolted a strong steel casting to form the meeting faces.

To obtain a perfect fitting between the quoin bearing piece and the bearing plate of the hollow quoin, provision has been made to bolt the hollow quoin bearing plates to the quoin plate and leave a recess in the concrete work of the hollow quoin to be filled in after the gate is in place. To secure perfect fitting at the miter, the miter post bearing pieces have a vertical slot, in which is placed a loose steel plate that can be moved horizontally by means of bolts. When the gates are in place the loose pieces can be brought in contact and the space behind them filled with babbitt metal. The miter and quoin meeting faces are slightly curved to avoid "nipping" at the edges.

The gate pivot is a steel forging with a spherical top set in a cast steel base imbedded in the concrete floor. A steel casting which holds a bronze bushing that fits the pivot is bolted to the bottom of the gate. The pivots for all gates are of the same size.

The upper hinge is the same for all gates. It consists of a steel pin securely fastened to the top of the gate and a cast steel collar bushed with brass.

The gate anchorages are the same for all gates and consist of two steel eye bars attached to the upper hinge collar, from which they extend back into the masonry to a frame work of steel beams imbedded in the concrete. In order to simplify the setting of the anchorages, the framework is made rectangular and will set hori-

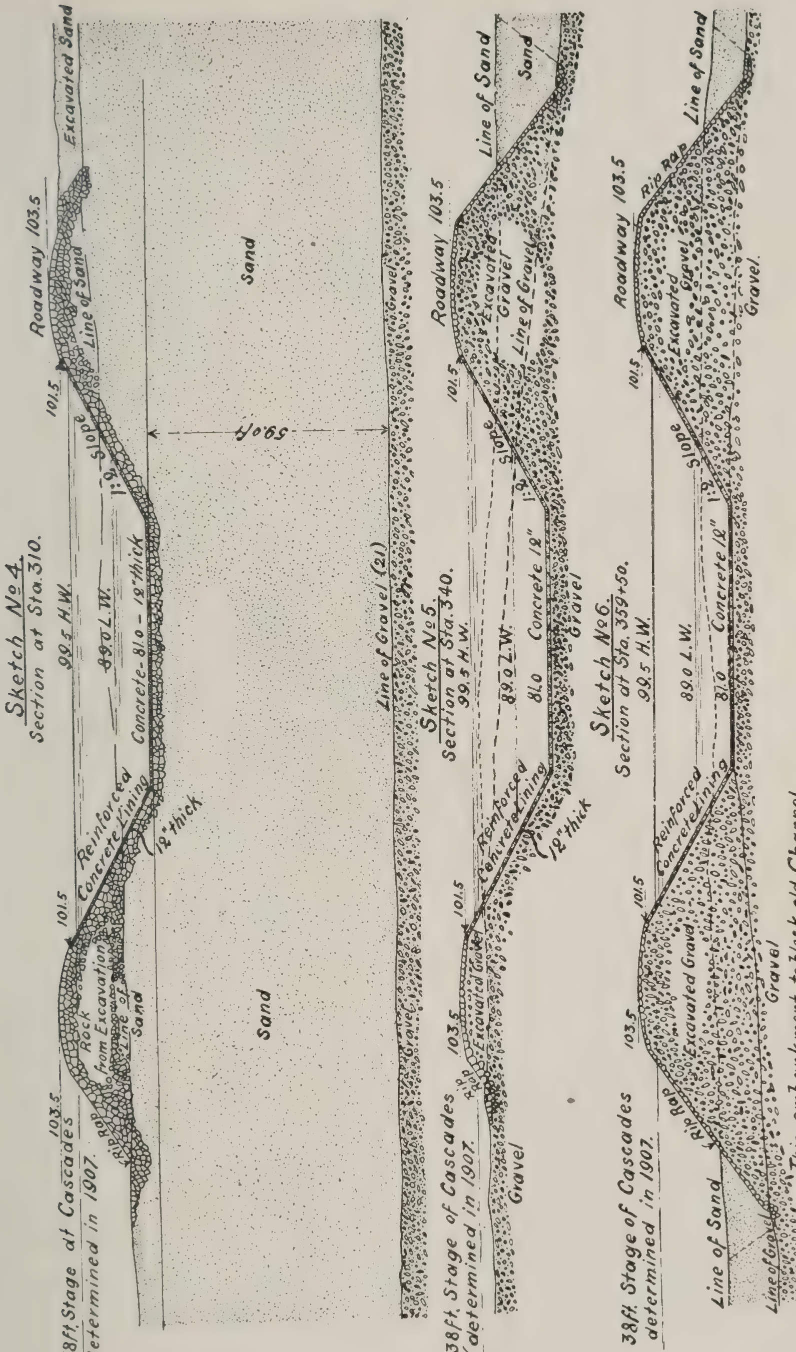


Fig. 7. Typical cross sections of canal in sand and gravel as actually being constructed.

zontally. A recess will be left in the concrete at the proper elevation to be filled in after the gates are set. Provision has been made for adjusting the anchor bars.

The miter sill contact is made by a flat steel plate bolted to the bottom girder of the gate, and a casting, having a horizontal slot to receive a timber bearing piece bolted to the miter sill.

The Celilo lock gates, Ten Mile lock gates, and the lower gate of Five Mile Lock are fitted with butterfly valves for filling and emptying the lock chambers. The valves are constructed of steel with cast-iron journals and will be operated from a foot-bridge on top of the gates by means of levers and rods.

The details of all of the gates will be, as far as possible, made the same, to enable the contractor to use the same patterns and templets for all similar parts of the different gates.

The design of the gates is based on data obtained from "Mitering Lock Gates," by Col. H. H. Hodges, Report of Board of Engineers for the Deep Waterways and other engineering works.

WATER POWER.

While the original plans did not contemplate electric or water power for the lighting and operation of the canal, a power site has been selected near the head of Five Mile Rapids, where a minimum head of about 14 feet can be obtained at high water and a maximum head of about 46 feet at low water in the river. A second power site was selected at Big Eddy where, at ordinary high water, a head of about 18 feet can be obtained, but for an extreme flood there would be no head; at low water a head of 70 feet can be obtained by means of a tunnel and shaft in solid rock. The amount of power that can be developed at these sites will depend upon the quantity of water that can be taken from the pool above Five Mile Locks without affecting navigation.

OPERATION OF CANAL, ETC.

Power boats will navigate the canal under their own power, and in order to facilitate the passage of boats going in opposite directions it is probable that an electrically operated system of semaphore signals will be used at the passing basins.

The canal terminals at Celilo and Big Eddy will have a landing place in the river at the entrance to the canal. At Celilo a direct-connection with the railroads and wagon roads is possible by means of an inclined roadway or track. At Big Eddy the trans-con-

tinental tracks are over 100 feet above the low water of the river and a direct connection with the boat landing would be difficult and unnecessary, for there will be no occasion to transfer freight from boat to rail or wagon at that point.

It is estimated that the canal will be in operation at least ten months annually. From the records of river readings it appears for a period of twenty-six years the river was above the limiting stage of navigation an average of twenty days annually, with a maximum of sixty-six days in 1880. During the twenty-six years it failed to reach the limit of navigation eight times.

The navigation of the river is likely to be suspended by ice during the winter months. Ice blockades generally occur in January and do not usually cover a period of more than three weeks. During the ice period the river is generally at a very low stage and navigation on the Snake River is usually suspended on account of low water, and navigation is then limited to the Columbia River between Celilo and Priest Rapids.

It is estimated that about two hours and fifty minutes will be required for the passage of a steamboat through the canal, moving at the rate of 4 miles per hour through the pools.

CANAL CONSTRUCTION WORK.

The first work in connection with the construction of the canal was the open river improvement of Three Mile Rapids about 1 mile below the canal entrance. To date there has been removed about 64,300 cubic yards of solid rock reefs and shore line, resulting in a channel with a least depth of 10 feet at low water and a minimum width of 200 feet. This work has been of great benefit to navigation through Three Mile Rapids.

The first canal construction work was begun in October, 1905, on a contract for the construction of ½ mile of canal at the upper entrance near Celilo. This contract, owing to the manner in which the contractors conducted the work, was not completed until May, 1910, although the time limit was January 1, 1907. Several extensions were granted to the contractors, who were required to pay the costs of inspection.

The second contract was let in September, 1908, for 14,000 feet of canal, with the exception of the concrete lining. The contractors finished the work within the specified time. About 20 per cent of the entire canal work was included in the above contract.

HIRED LABOR WORK.

Canal construction under the contract system was hardly satisfactory, and confined the construction work to such sections of the canal where the work could be completed for a sum within the amount of funds available, so that no unfinished work would be exposed to the river freshets.

The River and Harbor Act of June 25, 1910, appropriated $600,000 for continuing the work of canal construction with a view to completion within a period of six years, and it was decided to complete the canal work by hired labor.

The work remaining that could be done by hired labor was the construction of about 5 1-3 miles of canal (from the lower entrance at Big Eddy to Station 283), the completion of canal section from Station 294 to 263 by placing the reinforced concrete lining, the construction and installation of all lock gates, and the excavation for and construction of the canal entrance at Celilo. The quantities of excavation and masonry work to be done after the completion of the contracts was estimated to be as follows:

Solid rock excavation	870,000 cubic yards
Sand excavation	400,000 cubic yards
Common excavation	500,000 cubic yards
Concrete	150,000 cubic yards

In July, 1910, under authority of the Chief of Engineers, construction by hired labor was begun at the west or lower end of the canal. The work at first was rock excavation for the Tandem Locks at Big Eddy. Awaiting the arrival of the plant ordered, the work was carried on with such plant as could be obtained in the district. A few locomotives, derricks, and some tools were obtained from Coos Bay, and some from Cascade Locks—all old plant which had been stored for years but was still serviceable.

Camp construction began at once and by December, 1910, two camps were in use and most of the buildings at headquarters, Big Eddy, were erected. By April, 1911, there were three camps fully equipped for work and approximately 600 men at work on the hired labor section.

Many of the buildings constructed for use during the hired labor work are permanent buildings to be used as headquarters, lock keepers' quarters, shops, etc., after the completion of the canal.

Fig. 8 (upper). Completed rock section showing regulating weir to let flood waters into the canal before a general overflow. View taken with high water in river, about ½ mile below (west) of Celilo Lock.

Fig. 9 (lower). Completed section of canal, looking from canal lock wall upstream into the Celilo boat basin. Flood weir and tunnel entrance at left side of wall.

The plant in use by the hired labor force is as follows:

Three 60-ton steam shovels with 2-yard dippers.
One 40-ton steam shovel with 1½-yard dipper.
One 130-ton land dredge with 1½-yard dipper, a dumping radius of 65 feet, and a lift of 35 feet above rail.
Fifteen locomotives, 10 to 17-ton.
One hundred and seven 4-yard dump cars.
Thirty-six small dump cars for concrete work.
One rock-crushing plant complete.
One sand-grinding plant.
Two concrete-mixing plants.
Three air-compressor plants.
Thirty steam or air drills.
Nine derricks, complete with engine and boiler.
One electric-lighting plant.
Two deep wells, complete with pumping plant.
Thirty-one horses with wagons and scraper outfit.
Five extra boilers for steam drills and a complete machine, carpenter, and blacksmith shop for maintaining the above plant.

The sand grinding plant is run in connection with the rock crushing plant. The large rock is crushed to product, so that about 80 per cent will pass through a 2½-inch screen and the rejections from this screen are fed to a pulverizing machine which consists of a large number of steel hammers attached to a shaft by means of several links of strong chain. The shaft revolves at a rate of about 600 revolutions per minute, and the rock is pulverized by the hammers on a steel plate. About 70 per cent of the product of this machine will pass a ¼-inch screen and the remaining 30 per cent is passed through a set of crushing rolls. All the machinery for crushing the rock, grinding the sand, running the concrete plant and elevators is connected with one central boiler and one engine runs the several machines. The cost of the sand is small compared to the sand obtained from sand pits 100 miles distant and the quality is superior. The concrete plant is so arranged that the sand and broken stone is elevated from crushers into bins, fed into cars by gravity, and hauled up an incline to hoppers over the concrete mixer. The cement is stored in the warehouse alongside of the mixer house, and is elevated by means of an elevator to a platform above the mixer.

The mixed concrete is fed into V-shaped dump cars, on a track below the mixer, and these cars are hauled to the forms by locomotives. The longest haul from concrete mixer to forms was 7,000 feet and, as only a few minutes are necessary to make the run, the concrete arrived in good shape. The concrete is mixed rather dry, so that it will not settle into the cars enough to stick, and a small quantity of water is added to the mixture when it is dumped into

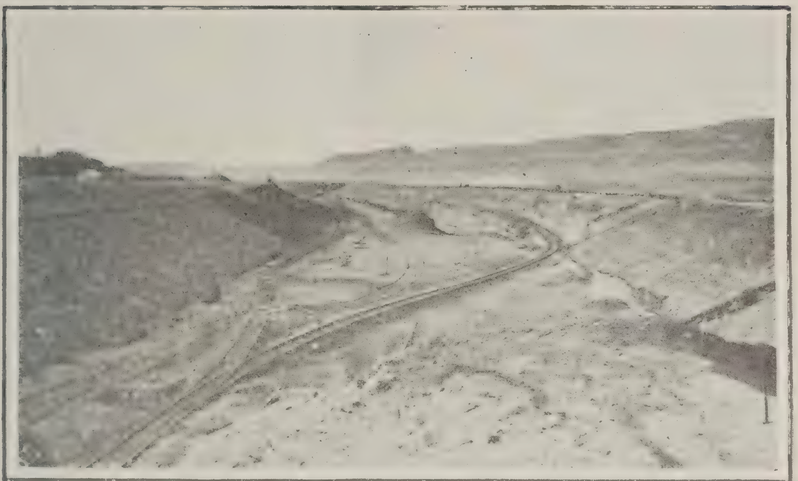


Fig. 10 (upper). Canal section in gravel showing paved slopes (both edges of picture) ready for concrete.

Fig. 11 (lower). Land dredge at work between stations 160 and 210, about half-way between Five and Ten Mile rapids. Note embankments being made with the excavated gravel. Dredge has 65-foot dumping radius and 35-foot lift above rail. Two-yard dipper used.

the bins from which it is fed into the push cars which take the concrete to the form in use. Various mixtures of concrete are used, depending mostly upon the quality of sand used. The usual mixture for the hired labor work is 1:3:6½.

Broken stone has been used exclusively on the hired labor work on account of its availability. The rock from excavations is loaded into cars by steam shovels and derricks, and is delivered to crushers without rehandling. There are extensive gravel bars along the line of the canal, but, while some gravel has been used for concrete, it is very probable that only broken stone will be used for concrete in the future, on account of the ease with which it is obtained from excavations with generally but little greater haul than to the dumps. No concrete work in the locks has yet been done by hired labor. The concrete work has been that of constructing canal trunk walls. These were built in blocks of from 20 to 40 feet in length, depending on the height of the wall. Alternate blocks are placed and the space between them filled when the first blocks have set. The studding and lagging are set up for a continuous form and bulkheads placed where desired. The joints between the blocks are marked by a V-shaped strip. No separate facing is put on the finished side, the coarse aggregate being spaded back from the frames for that purpose. Rock excavation is done by blasting and removal of shattered rock by means of derricks and steam shovels. In the deep lock excavations derricks are used and for canal trunk steam shovels are in use. The rock excavations are roughly made by heavy blasts and steam shovel or derricks, and then the excavation is widened to prescribed lines by means of light blasts, using pneumatic hammer drills for drilling the blast holes.

Where the canal is in gravel the excavation is made by a 130-ton land dredge, which has a 1½-cubic yard dipper, a dumping radius of 65 feet and a dumping lift of 35 feet above the rail. The dredge excavates one-half the canal section without lateral shifting of track and works ahead for a long stretch, after which it is backed up and takes out the remaining half of the excavation. The long boom enables the dredge to build the embankments for the canal, generally without the use of cars to haul the material. In cases where it is necessary to haul some of the excavated material, the dredge can load the cars on top of the embankments, thus eliminating the long haul up steep grades from the bottom of the excavation. It is estimated that the use of this land dredge has reduced the cost of gravel excavation in this canal work one-half.

Fig. 12 (upper). Excavation for tandem locks flooded during high water in the Columbia.

Fig. 13 (lower). Completed canal section looking east from boat basin near Big Eddy, only a short distance from lower edge of upper figure.

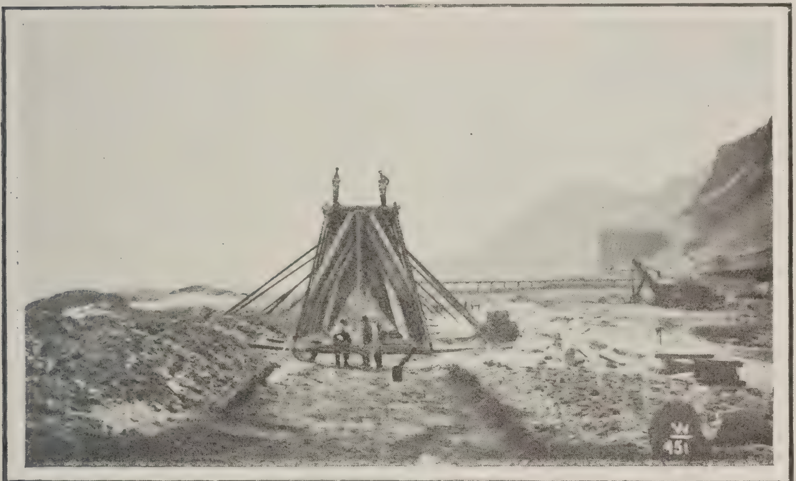
Control of the sand dunes has been one of the problems in railroad operation through this section of the country, the railroads contenting themselves with sand fences, which are effective so long as the wind blows from one direction. However, crude oil is now being experimented with. Along this portion of the river the wind usually blows upstream and the sand fences are set accordingly, although a strong East wind, which usually occurs during the winter months, will, if the sand is dry, completely stop railroad operations in a few hours. The sand dunes along the canal line are being leveled off and covered with rock from excavation, which will effectually stop their movement. The possibility of increased sand deposit by high water will only occur during the very highest freshets, as it is proposed to make the river embankments high enough to exclude all but the highest freshets. The deposits of sand by the lower high waters can be controlled before it reaches the canal embankments. The deposit of sand by very high floods is not to be feared so much as that during moderate freshets, for, during extreme high stages, the slope of river in the vicinity of sand deposits along the canal is very flat and will not carry very much deposit. The most troublesome shifting sand along the canal is that which has been deposited by ordinary floods and later blown to high ground out of reach of the floods.

It is also proposed to control the sands to windward of the canal by planting willows, grasses, and grains, as has been successfully done in many places along the river.

ORGANIZATION OF HIRED LABOR FORCES.

The administration of the affairs for the field work of the hired labor forces is carried on at canal headquarters building at Big Eddy under the direction of Capt. H. H. Robert, Corps of Engineers. The force consists of one assistant engineer, two junior engineers, two surveyors, who also act as draftsmen, a mechanical draftsman, one chief clerk, a cost-keeping accountant, one stenographer, one head timekeeper, a cement tester, a general storekeeper. The hired labor forces consist of two complete camps with a superintendent in charge of each camp, and the necessary overseers, timekeepers, cooks, etc. The superintendents have charge of the camps and construction work and report to headquarters, from which all orders are issued and all accounts are kept.

In order to keep an accurate account of the cost of the work, a cost system was begun as soon as the hired labor work was started, and was afterwards elaborated to meet the demands of the more ex-



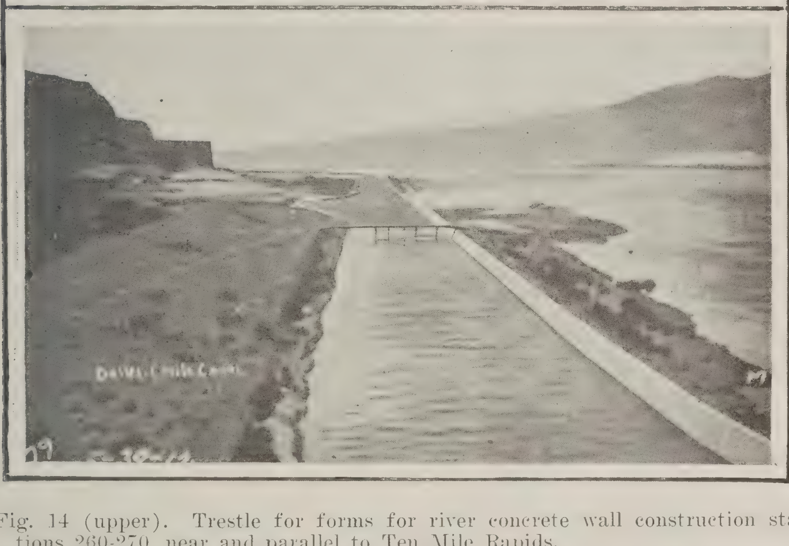


Fig. 14 (upper). Trestle for forms for river concrete wall construction stations 260-270, near and parallel to Ten Mile Rapids.
Fig. 15 (lower). Completed section of canal, looking west from station 410, about ½ mile west of Celilo Lock.

tensive work. At first the cost accounts were made up from the foreman's daily report of labor and material used, but later a cost system similar to that in use by the United States Reclamation Service was adopted. In this system every employee and every item of work is given a number, and a careful account is kept of the cost of each item of work for material and labor. This necessitates triplicate requisitions for all materials and supplies, so that the record of the amount issued and of that delivered can be kept, also careful inventories of supplies on hand and of that used must be kept. A careful record of each employe's time is kept and the nature of his work during each one-fourth day, so that the same can be charged to the proper item. At the end of each calendar month the data is compiled, which results in the unit cost of each class of work and each item relating to that class. The accuracy of the unit cost depends entirely upon the amount of care taken in distributing the cost of the material and labor to each particular item, and what credits are allowed for the material on hand. The unit prices for different pieces of work will vary from month to month, depending much on the manner of distributing the cost, as material charged near the end of one month may not be used until the following month, and unless credit be given for the material on hand the first month will show a high unit cost and the following month a correspondingly low unit cost. The actual unit cost of doing any one piece of excavation or concrete work can not be determined until the work is finished, but the monthly unit cost will serve as a guide for future work.

The laboratory work has been confined principally to the testing of cements and mortars, but it is the intent to install apparatus for making all necessary tests of materials used on the work. The progress of the hired labor work has been excellent and the results satisfactory, as there is opportunity to make minor changes of the plans which will result in a saving of cost without considering the effect it would have on a contract with definite plans and specifications. Heretofore, while all of the contract work was let on a unit price for different classifications, any desired change which would result in economy for the United States would meet with a protest on the part of the contractors if it did not meet with their approval.

On account of lack of sufficient funds to carry on the work, it was necessary, about October 1, 1911, to reduce the working force to such an extent that the plant was not worked to its full capacity and forces were reduced month by month so that by June 1, 1912, the work was closed down, awaiting further appropriations of

funds. Work did not resume until after the passing of the River and Harbor Bill, approved July 26, 1912.

The following table shows the progress of the canal work by contract and hired labor since the beginning of the canal construction work to the close of the calendar year ending December 31, 1912.

Total quantities of material removed by contract and hired labor.

Material moved and placed, etc.	Contractors Smyth & Jones. 53 months.	Contractors Caughren, Winters, Smith & Co. 31 months.	Hired labor. 30 months.	Total to December 31, 1912.
	Cubic yards.	*Cubic yards.*	*Cubic yards.*	*Cubic yards.*
Solid rock excavation	70,836	360,019	513,560	944,415
Sand excavation	9,156	340,261	375,285	724,702
Common excavation	35,644	176,953	268,470	481,067
Masonry	33,338	15,928	27,742	77,018
Riprap	6,314	8,322	3,740	19,009
	Linear		*Linear.*	*Linear.*
Tunnels	450		1,215	1,631

Smyth and Jones began work in October, 1905, and finished February, 1910.

Caughren, Winters, Smith & Co. began work in November, 1908, and finished May 30, 1911.

Hired labor work began in July, 1910.

On January 1, 1913, it was estimated that 60 per cent of the construction work had been completed.

It was estimated that, if sufficient funds were available, the canal could have been opened to navigation about January 1, 1914, but owing to the lack of funds necessary to carry on the work at a maximum rate during the past fiscal year, the opening of the canal, should there be no further delay in securing necessary funds, will take place during the fall of 1914, or not later than January 1, 1915.

The opening of the canal to navigation may not affect the existing railroad rates to up-river points for the reason that since the installation of the State Portage Railroad to Celilo and the steamers put on the river by the Open River Association, the railroads have several times reduced the freight rates to points above the Dalles; with the canal in operation there will be opportunity for competition, but until the commercial resources of the country tributary to the river are developed it is doubtful if the commerce on the river will attract competition.

There are now only two steamers operating on the river above Celilo in connection with the Portage Railroad, and it is understood that these boats are operated at a loss, but as this company is composed of business men, of Portland and the inland cities, who are

large shippers to points tributary to the river, the loss is compensated for by the reduction on railroad freight rates. To revive the navigation of the river above Celilo will require a number of serviceable boats and men to handle them. There are only a few men who are familiar with the Upper Columbia and Snake rivers at all stages, and to navigate the rivers at low water requires an intimate knowledge of the river and considerable skill. Most of the old navigators have passed away, and as there has been practically no navigation of the Upper Columbia and Lower Snake Rivers for the past twenty years, the knowledge of the old captains was not handed down to apprentices as in the case of rivers with continuous navigation. The risks and cost of operating steamers on the river above Celilo will be much greater than for the middle river, and the effect of the opening of the canal will therefore be felt more in the regulation of freight rates than by the commerce of the river.

Fig. 1. Electric device (used to actuate the camera for all work of this kind) placed so as to operate shutter of camera as soon as mortar began to recoil. Made use of the ''break'' method, having a spring to assist the demagnetization of coils.

Views of Mortar Shell in Flight*

BY

Capt. F. J. BEHR
C. A. C.; Instructor Coast Artillery School

The first four prints are those of trial shots and the last three those of record shots. As can be seen from the shadows, the trial shots were taken near noon while the record shots were not fired until after 4.30 p. m. of the same day, when the entire mortar pit was shaded by the trees in rear of it. Consequently, there are no shadows on the last three prints. The delay between the trial and the record shots was due to shipping in the vicinity of the targets. All the prints were taken with the same exposure, 1-5,000 of a second. Considering the lateness of the hour, the fact that the entire mortar pit was in shadow, and the very rapid exposure necessary, the definition obtained is remarkable.

For the purpose of obtaining views of guns and mortars in action, both the "make" and "break" methods were used for actuating the electrical device that operated the shutter of the camera. The electrical device was also arranged to remain fixed and then employ the chronographic principle for obtaining exceedingly small decrements in change of time. The recoil of guns and mortars, as was the case with this set of pictures, have been used to operate the system. The projectiles themselves have likewise been used for the same purpose, eliminating the variable recoil and elasticity of the carriage to which the device may be attached.

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*The accompanying pictures are republished by courtesy of the *Artillery Journal*, in which they first appeared in the September-October, 1912, issue.

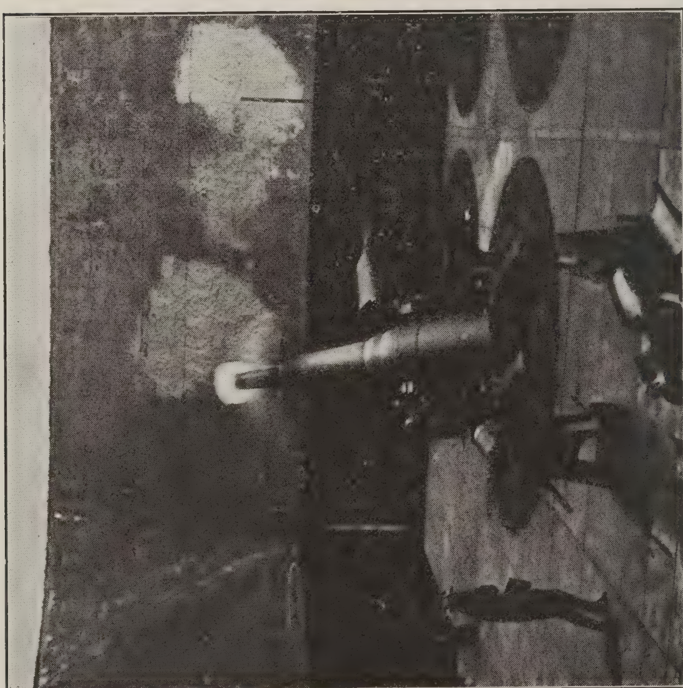

Fig. 2. Device set about ¼ inch later than No. 1. From observation made so far in taking photographs of gun and mortar discharges, it is believed that the ring of gas around and just in rear of the cap of this projectile is due to improper ramming, although there is not sufficient data available at present to substantiate this assertion.

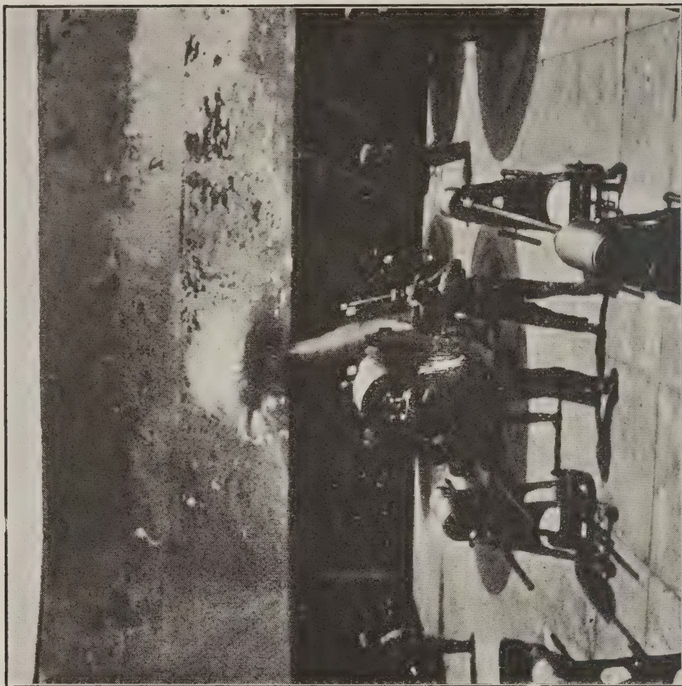

Fig. 3. Same as No. 2, except setting of device not quite ½ inch. Shows first step in formation of smoke cone.

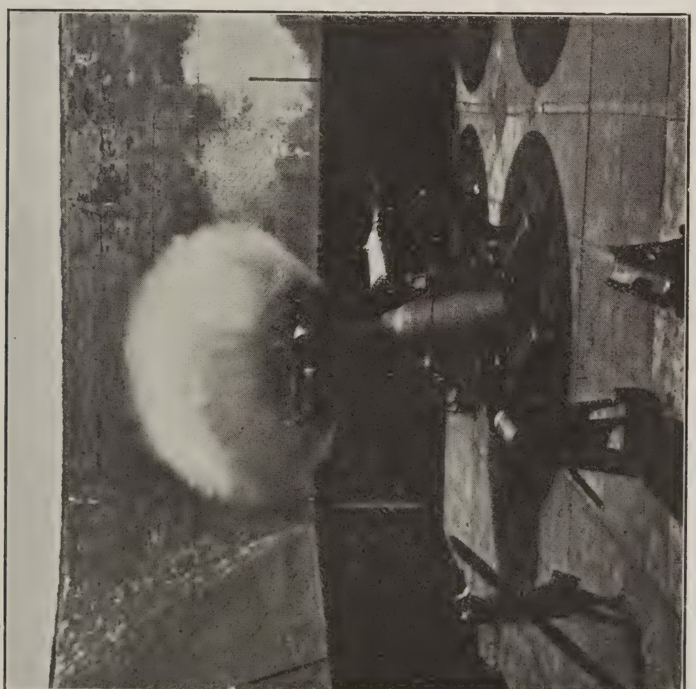

Fig. 4. About 1-10,000 second later than Fig. 3—setting on recoil.

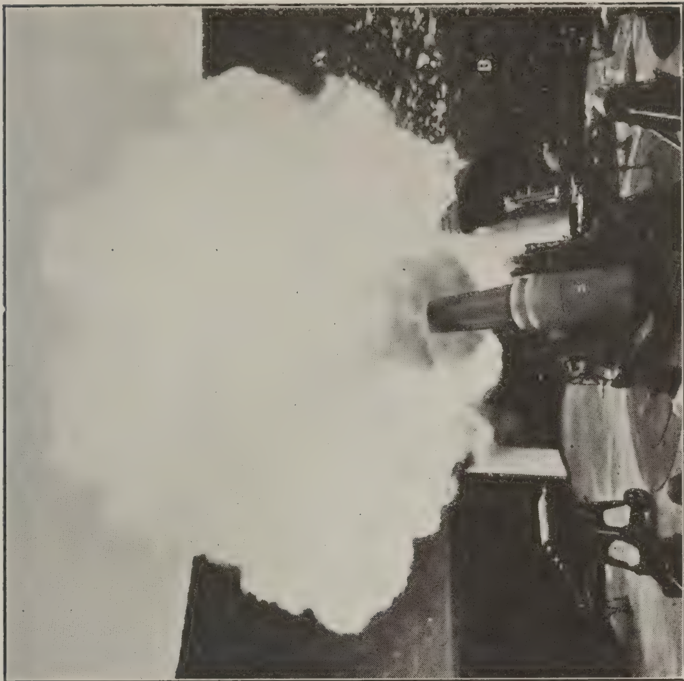

Fig. 5. Setting about 1 inch. This effect is what is usually seen first by the eye.

Fig. 6. Setting could not be accurately determined on account of not having sufficient time; and not having enough plates on hand at the battery when the firing was taking place to continue the small increments in change from previous settings.

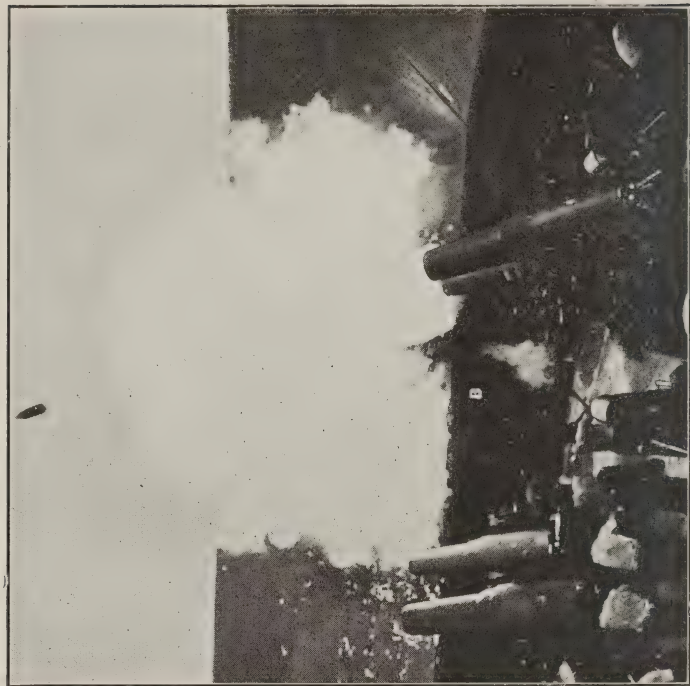

Fig. 7. Setting was made to obtain late effect. This is not as difficult as the first few settings, where due care must be exercised to allow for the smallest fraction of time.

Control of River Floods*

BY

Col. C. McD. Townsend
Corps of Engineers; Member American Society Civil Engineers

The subject of land drainage is intimately associated with that of river improvement. The cultivation of the soil largely increases the amount of sediment entering our streams; the direction of the furrow markedly affects the amount of rainwater that flows from its surface, and every ditch or subsurface drain promotes a more rapid flow into our rivers during floods and possibly affects their discharge during low water. On the other hand, no satisfactory system of land drainage can be accomplished in a country subject to periodic overflow by river floods. In the Mississippi Valley protection from floods is absolutely required before any regular system of drainage can be inaugurated. The overflow is so great and the amount of sediment carried by the river so large, that the drains would be annually destroyed or filled. The floods not only insure the destruction of any crops that might be planted, but also usually occur at such times as to prevent the harvesting of a second crop the same year. A discussion of the means of preventing floods in the Mississippi Valley is therefore particularly appropriate at this meeting. In a paper read before a levee convention in Memphis last September, I briefly discussed the various means of flood control which had been suggested to the Mississippi River Commission. This afternoon I propose to confine my remarks to the three methods in which the public appears most interested—that is, reforestation, reservoirs, and levees.

SOURCES OF FLOODS.

Before entering upon such a discussion, it is desirable to have a clear conception of the sources from which floods arise.

As you will recall, the greater Mississippi Valley is bounded on the east by the Appalachian chain and on the west by the Rocky mountains. These mountain ranges exert a great influence on its

*Paper read at a recent meeting of Drainage Congress in St. Louis.

floods. The winds blowing from an easterly direction deposit most of the moisture they absorb from the Atlantic Ocean on the eastern slope of the Alleghenies, and therefore cause little rain in the Mississippi Valley; the Rocky Mountains intercept the moisture from the Pacific Ocean. While showers occur from winds blowing over the Great Lakes, the original source of the floods of the Mississippi is to be sought in the Gulf of Mexico.

During the winter and spring the land of the Mississippi Valley, no matter what its soil or the nature of its covering, is cooler than the waters of the gulf, and a southerly wind becoming saturated with moisture as it passes over the water will precipitate that moisture on the land in copious rains, or in snow when the temperature is sufficiently low. A wind from the southwest sweeps up the Ohio Valley; one from the south carries moisture to the Upper Mississippi; one from the southeast to the valleys of the Arkansas and the Missouri; but in all cases there is a tendency for the greatest rainfall to occur near the coast, and gradually to decrease as the wind currents travel inland. Thus the average annual rainfall at New Orleans is 60 inches; at Memphis, 52 inches; at Cincinnati, 42 inches; at Pittsburg, 36 inches, and at St. Louis 40 inches. At the headwaters of the Upper Mississippi it is but 25 inches, and at the headwaters of the Missouri but 13 inches. Though floods do not arise from mean conditions, but from exceptional rainfall when 6 to 10 inches fall in a week, these figures are good indices of flood volumes, as we find from observation by the geological survey at Williston, N. D., that the flood discharge of the upper reaches of the Missouri is about one second-foot per square mile of drainage area; measurements at St. Paul give an extreme flood discharge for the Upper Mississippi of slightly over 2 second-feet per square mile. In the Ohio it is about 6 second-feet and in the Ouachita, St. Francis and Yazoo rivers from 8 to 10.

From the above it will be seen that the rainfall is very unequally distributed over the Mississippi Valley, being least at the upper sources of the tributaries, and rapidly increasing as you approach the main stream, though an exception is to be noted in the southern tributaries of the Ohio, whose sources are nearer the gulf than are their outlets.

The maximum discharge of the Upper Mississippi River is estimated at 450,000 second-feet; the Missouri, 900,000; the Ohio, 1,400,000; the Arkansas, 450,000; and the Red, 220,000. There is also a large discharge from the Yazoo, St. Francis, White, Tensas,

and Ouachita rivers. The maximum discharge of the Mississippi during the flood of 1912 was about 2,000,000 second-feet at Cairo, and 2,300,000 at the mouth of Red River. It overflows its natural banks when the flow exceeds 1,000,000 second-feet.

While the influence of forests on stream flow has received little attention in this country until recently, the scientists of Europe have discussed the subject pro and con during the past forty years. It is generally accepted by both sides that the leaves falling from forest trees as they decay form a humus which has a large capacity to absorb water, and that when the forests are felled this humus is seriously injured by forest fires. It is also admitted that snow is more rapidly melted when it is exposed to the direct rays of the sun in an open field than when sheltered from such action in a forest. In fact, it has been found by the United States forestry service from experiments recently made in the White Mountains that the flow from cleared fields under such conditions is about twice that from forests. The forest advocates claim that this is sufficient proof that forests absorb water during flood periods, which percolates through the ground and flows from springs later in the season, thus reducing flood heights and increasing the low water flow of rivers. Its opponents do not admit that the problem is thus easily solved. They claim that floods do not arise from the melting of snows by the direct action of the sun; that this process is so slow that the water which flows off would not raise a river to mid-stage; that floods occur when on a layer of snow there falls a copious supply of rain, and both the rain and melted snow enter the stream simultaneously; and that under such conditions the forest, instead of being beneficial, is injurious. On cleared land the wind tends to blow the snow from the ridges and piles it in immense masses in the ravines, while in the forests the snow is uniformly distributed. A few days of sunshine dries out the ridges in the open field and melts sufficient snow in the forest to saturate with water the underlying humus.

If a heavy rainfall then occurs, the forest humus, being saturated, can absorb no more water, and the combined rain and snow of the forest flows into the streams, while in the cleared land, the ridges having dried out, absorb a large portion of the rainfall, and the snowdrifts expose a much smaller surface to the action of rain. Moreover, during periods of great drouth the forest humus and long, deep tree roots also absorb more water than grass and farm crops, and retard the run-off at a time when it is most needed for low water navigation. They therefore maintain that a forest is a

fair-weather friend of some use in regulating the mid-stages of a river, but an utter failure when most needed—that is, during extreme floods or extreme low water. While I consider this discussion valuable, my objections to reforestation are not based solely on a scholastic argument.

It requires from twenty to fifty years to produce a good forest growth, and over a century for the leaves of that forest to decay in sufficient quantities to produce the humus which will be satisfactory as an absorbent of rainfall. We can not afford to delay the drainage of the Mississippi Valley even to produce the forest growth, without taking into consideration the time required for the humus to form. We are more vitally interested in the height that the river will attain in the next few weeks than in what will occur in the year 2013.

It is also pertinent to this discussion to determine what would be the extent of the forest reservation which would be required to reduce the flood heights on the Mississippi River a given amount. To solve this problem it is necessary to make certain assumptions, and for purposes of argument we will take it for granted that reforestation will reduce the flood discharge of a stream one-half. The Mississippi flood of 1912 attained the greatest height of any then recorded at all gauge stations, except at Vicksburg. That of January and February, 1913, while 5 feet lower at Cairo, was the next highest flood at Memphis, and for a considerable distance along the river. We will endeavor by reforestation to reduce the flood of 1912 to the heights attained in the winter of 1913. For this purpose it will be necessary to reduce the maximum discharge of the river 500,000 second-feet. It will also be necessary to distribute this reduction among the tributaries, reducing the maximum discharge of the Missouri River from 900,000 to 700,000 second-feet, that of the Upper Mississippi from 450,000 to 350,000, and that of the Ohio River from 1,400,000 to 1,200,000.

As stated in the introductory remarks, the flood discharge of the Missouri River at its headwaters is about 1 cubic foot per second per square mile of drainage area, and if the reduction in discharge of one-half is to be secured by reforestation 2 square miles of forest would be necessary for every second-foot of reduction of flood discharge, or 400,000 square miles of forests to reduce the discharge of the Missouri River 200,000 second-feet. At the headwaters of the Upper Mississippi the ratio of flood discharge to drainage area is about 2 second-feet per square mile. A reduction

of this discharge by one-half would require a forest reservation of 100,000 square miles to reduce the floods of the Upper Mississippi 100,000 second-feet. On the Ohio River the ratio is six to one, and it would therefore require forests at the headwaters of the Ohio having an area of 66,000 square miles to reduce its flow 200,000 second-feet. In other words, to reduce the height of a flood at Memphis by reforestation at the headwaters of the river from that of 1912 to the next highest on record would require a forest reservation of about 566,000 square miles, an area exceeding that of the portions of Montana and Wyoming drained by the Missouri River and North and South Dakota, the portion of Minnesota drained by the Upper Mississippi River, and the States of Wisconsin, Iowa, Illinois, and Indiana. But even such a forest reservation would afford only partial protection, and large expenditures for levees would still be required. Under the above assumptions, to prevent any overflow by reforestation would necessitate a practical abandonment of the valley for agricultural purposes, and the development of an extensive irrigation system to produce tree growth in arid regions of the west.

It is therefore apparent that even under the most extravagant claims of forestry advocates, reforestation as a means of reducing flood heights on the Mississippi requires the conversion of too much farming land into a wilderness to be practicable. The waste land that can profitably be converted into forest reservations is too limited in area to produce an appreciable effect on the floods.

RESERVOIRS.

To have retained the Mississippi flood of 1912 within its banks would have required a reservoir in the vicinity of Cairo, Ill., having an area of 7,000 square miles, slightly less than that of New Jersey, and a depth of about 15 feet, assuming that it would be empty when the river attained a bank-full stage. If the site of such a reservoir was a plane surface, the quantity of material to be excavated in its construction would be over 100,000,000,000 cubic yards; and its estimated cost from 50 to 100,000 million dollars. Such a volume of earth would build a levee line 7,000 miles long and over 150 feet high.

Cairo is the logical location for a reservoir to regulate the discharge of the Lower Mississippi. It will not only control the floods from the Ohio, but also the discharge from the Missouri and Upper Mississippi. But if the reservoirs be transferred from the mouths

of the tributaries to the headwaters, their capacity must be largely increased. No two floods have the same origin, unless they are referred back to the Gulf of Mexico. The wind bloweth where it listeth. If the prevailing winds in the early spring are from the southwest, the southern tributaries of the Ohio furnish the crest of the year's flood; if more nearly from the south, reservoirs will be required on the streams of Ohio, Indiana, and Illinois; a slight varying of the wind will produce a flood in the Upper Mississippi, while if it blows from the southeast the principal sources of trouble will be the Red, Arkansas, and Missouri rivers. To control the flow of every stream in the Mississippi Valley by reservoirs is a pretty large job, even for the United States Government, but that is what the control of the Mississippi during floods by reservoirs signifies.

The advocates of the control of the floods of the Mississippi by reservoirs do not, however, have in mind any such radical control as is above indicated. They limit the control to the headwaters of the various tributaries, and while every stream that flows in the valley may be considered a headwater of some tributary, I judge from the discussions of the reservoirs and their proposed employment for power purposes, which requires a considerable height of dam, that by headwaters is meant the sources of the rivers in mountainous countries as distinguished from the more level plains, and more specifically the sources of the Missouri above the mouth of the Yellowstone, those of the Upper Mississippi in the State of Minnesota, and those of the Ohio in the Appalachian range.

The flood which is now devastating the country affords data for determining the effect of such a system of reservoirs and its lessons are the more valuable because no effort is necessary to refreshen the memory. When, on April 2, the gauge at Cairo attained a height of 54 feet there was flowing down the Mississippi River at least 2,000,000 cubic feet of water per second. It requires about eleven days for a flood wave to be transmitted the 966 miles between Pittsburgh, Pa., and Cairo; on March 22, the Pittsburgh gauge read 5.3 feet, which is produced by a flow in the Ohio River at that locality of about 15,000 second-feet. In ten days a flood travels the 858 miles between St. Paul, Minn., and Cairo; on March 2 the reading of the St. Paul gauge was 0.5 foot, corresponding to a discharge of the Mississippi of about 2,500 second-feet. In eight days the effect of a flood at St. Joseph, Mo., is felt at Cairo; on March 25 the gauge at St. Joseph read minus 0.1 foot, representing a discharge of the Missouri River of about 17,000 second-feet. If a system of reservoirs had been constructed which would have prevented all flow from the Allegheny, the Monongahela, the Mississippi above St. Paul and the Missouri above St. Joseph, it would have reduced the 2,000,000 second-feet discharged by the Mississippi River at Cairo on April 2 less than 35,000 second-feet.

The water which passed Cairo on the 2d of April came principally from the White and Wabash, and the lower tributaries of the Ohio,

and after the water of these rivers started to subside the flood from Cincinnati, though increasing from 57 to 69 feet on the gauge, could increase flood heights at Cairo less than 1 foot. The flood of 30 feet at Pittsburg on March 28 produced its effect on the Cairo gauge day before yesterday (April 8). It has merely prolonged the flood without increasing its height.

The proposed system of reservoirs would have cost hundreds of millions of dollars and its effect on this year's flood height of the Lower Mississippi could not possibly have exceeded 6 inches.

Neither the rain nor snow which falls upon the mountainous portions of the Mississippi watershed has much effect upon the floods of the lower river. The principal source of the floods is the great alluvial plain between the mountains. As I have pointed out, excepting the southern tributaries of the Ohio, the rainfall is relatively slight at the upper sources of the tributaries and their maximum flood discharge does not usually coincide with that of the mid-valley.

Great floods do not arise from average conditions, but from exceptional variations such as are caused by a series of heavy rains rapidly succeeding one another. Each rainstorm starts down a stream a flood, the volume of which can be absorbed by a reservoir with comparatively little trouble, but if a second storm sweeps over the valley the reservoir to be effective must be emptied or its capacity doubled. To hold all the excess rainfall till low water would require reservoirs of enormous capacity. Economic considerations usually require that the reservoirs should be emptied as soon as the crest passes, in order to utilize the same space for a second rainfall; so that while reducing the crest of a flood at a given locality they necessarily prolong the period during which the river remains at a high stage.

The water which is abstracted from the Gulf of Mexico is usually precipitated in the Mississippi Valley within a period of two days. The return flow extends over a period of two or three months. The sum of the maximum discharges of the various tributaries of the Mississippi River is nearly 4,000,000 second-feet, while the greatest measured discharge of the river itself is about 2,300,000. This apparent discrepancy arises from the fact that the floods of the tributaries do not reach the gulf at the same time. The crest of the Ohio River flood usually passes down the river in March or April, that of the Missouri in May or June. Moreover, the same law applies to the tributaries of a tributary. The waters of the southern branches of the Ohio tend to discharge into that river before those in Ohio, Indiana, and Illinois.

By the construction of reservoirs, this beneficent law of nature is deranged. Instead of the crest of the flood of one stream passing down the river before that of another reaches it, two prolonged high stages will obtain which will tend to synchronize and the resultant combination may be higher than either flood would have been by itself.

A system of flood control designed to be satisfactory for one city may be most disastrous to another locality further down stream. If a system of reservoirs had been in operation in the Allegheny and Monongahela rivers during last January, it would have protected Pittsburgh from overflow and diminished the flood at Cincinnati when it was 50 feet on the gauge, but only to increase it when it attained a height of 60 feet. And this effect would have been propagated to the gulf.

Pittsburgh, moreover, would never consent to such a manipulation of reservoirs on the upper tributaries of the Ohio as would insure the reduction of floods at Cincinnati or on the Lower Mississippi. Source stream control on the Mississippi River and its tributaries would therefore soon reduce itself to the question, Whose ox is to be gored?

LEVEES.

While the use of forests or reservoirs as a means of flood control is still in an experimental stage all over the world, the employment of levees for this purpose has been tested for centuries. The Po, Rhine, Danube, Rhône, and other rivers of Europe have been successfully leveed. The laws governing the flow of water in a confined stream have been carefully studied and the construction of levees is just as susceptible of mathematical analysis as other engineering problems. There is an element of uncertainty in all the forces of nature. No one can say positively, for instance, that St. Louis may not at some future time experience an earthquake or a cyclone of greater intensity than that which swept the city in 1896. There is also a possibility that there will be some combination of meteorological conditions which will create a flood of greater volume than has heretofore occurred in any drainage area. But the height to which levees should be constructed is as susceptible of determination as the strains to be permitted in an office building due to wind pressure or the moving load allowable on a bridge. The city engineer solves a similar problem whenever he constructs a sewer to carry off the storm water from the city streets.

Nor is there any evidence either that floods have been increasing in recent years, due to the cutting off of forests or that the beds of our main rivers are rising as they are leveed. The effect of forests on rainfall in Europe has been carefully investigated by Profs. Schlichting and Hagen. The records at London, Paris, St. Petersburg, and other localities where the rain has been recorded for long periods fail to show any tendency to an increased fall in recent years.

The meteorological records of the United States have not been maintained a sufficient length of time to be of much value in solving the problem. Such data as we possess indicate that the flood discharge has not increased in recent years. The greatest flood of the Mississippi at St. Louis occurred in 1844, the next largest in 1875. On the Great Lakes the high water of 1838 is the greatest on record. In the Ohio the flood of 1884 exceeded that of

1913 at Cincinnati, and that of 1832, while 5 feet lower at Cincinnati, was 5 feet higher at Pittsburg than this year's flood. The gauge records at the bridges over the Upper Mississippi, which cover a period of thirty years, would indicate that the flow from Minnesota and Wisconsin, where the forests have been most extensively destroyed during the period, has been slightly improved, though the river shows signs of deterioration where it receives the flow from the prairie lands of Iowa and Illinois.

On the Merrimac River where the mill owners have accurate observations extending back to 1849 there appears to be some increase in flood discharge, though reports of the forestry commission of the State of New Hampshire and census returns from the State of Massachusetts indicate that the forest area of its basin is 25 per cent greater than forty years ago, due to the abandonment of farms. Such records as we have in this country appear to confirm the conclusions of the German forestry authorities that the influence of forests on drainage is concealed by other causes more powerful in their effects.

The flood of 1912 was not due so much to excessive precipitation as to the fact that the surface of the ground was still frozen over the States of Illinois, Indiana, and Ohio, so that there was not the soil absorption of rain water that usually occurs.

There is not the remotest connection between deforestation and the disasters which have just occurred at numerous cities in Ohio and Indiana. The flood of 1832 was similar to that of 1913, but it was discharged by streams of the dimensions the Creator intended they should have. Since then cities have sprung up and land has become so valuable that riparian owners have encroached upon the waterways. Where the floods formerly flowed untrammeled, factories and dwellings have been constructed and numerous bridges have further restrained the stream's discharge. When His laws are violated, though slow to anger, the Creator occasionally asserts His might and the works of man crumble before Him.

If it would accomplish any useful purpose, I could name other cities where conditions are as dangerous as at Dayton or Columbus, but the lessons of the flood will be forgotten with the burial of its dead.

The question of the rise of the river bed by levee construction has been exhaustively investigated. On the Rhine the maximum effects were observed at Dusseldorf, where the same discharge at low water appears to attain a height 8 inches greater to-day than it did one hundred years ago, while the same discharge at high water has lowered about 1 foot in a century. On the Po the maximum observed change in low-water conditions was 0.02 of a foot per year, but it is by no means proved that even these small changes have resulted from levee construction. They may have arisen from the improvements in the river bed which were made simultaneously with levee construction. The observations of the Mississippi River

Commission agree with the Dusseldorf observations, in that the Mississippi River appears to be slightly enlarging its section, at least at mid-stages.

The present contents of the adopted levee line of the Mississippi River is about 243,000,000 cubic yards. It has been computed that with an addition of 200,000,000 cubic yards and at an estimated cost of $57,000,000 this line would be safe against any flood which has occurred in the Mississippi River. This sum, though large, is less than $4 per acre of land protected from overflow, and appears insignificant when compared with the amounts which are being expended per acre for irrigation purposes in the arid west. The increase in the value of land or the damage caused by one overflow like that of 1912 would pay for the completion of the levee system.

When a levee line contains but one-half the material that safety requires, crevasses afford no argument against levee construction. During the flood of 1912 hundreds of miles of levees were topped with earth in sacks to a height of from 2 to 4 feet, to prevent the water flowing over them, and water was seeping through their narrow bases in copious streams, which was unheeded until mud began to flow. The levee which failed at Beulah, Miss., this winter was held till the pile of sacks exceeded 20 feet in height.

The holding of 1,525 miles of such levees through the flood of 1912, even though 13 miles failed, is a powerful argument in favor of a properly constructed levee line. There was no failure where levees were built to a suitable grade and adequate dimensions, as in the Upper Yazoo District.

CONCLUSION.

While of the opinion that levees afford the only practicable method of controlling the floods of the Mississippi River, I desire to state that I am strongly in favor of both reforestation and reservoir construction, but limited to the purposes for which they are adapted, just as I am in favor of reinforced concrete for small bridges, though not considering it applicable to one spanning the lower Mississippi River. The price of lumber to-day is a sufficient argument for planting trees, without attempting to associate forestry with the climate or with the flood conditions on our rivers. If the Federal Government or the states do not conserve the forests, the time will soon come when the farmer will raise his crop of timber just as he now plants a field of wheat, and for the same reason, because it will pay him to use his waste land for the purpose.

Reservoirs are necessary for municipal water supplies, for purposes of irrigation, for the development of power and for feeders to canals. They can be successfully employed on small streams to diminish floods or increase the low-water flow. The trouble arises when an attempt is made to utilize them for too many purposes at the same time. There must be a paramount issue to which the others will be subsidiary.

If the main purpose is to supply a city with water only the excess can be used for power development. If the dams are constructed to produce power, the reduction of floods and the improvement of river navigation must be subordinate thereto. Water required for irrigation can only be used to develop power when the dam of the storage reservoir is given a greater height than is necessary for its flow over the land to be reclaimed.

During the next decade there will be an enormous development of reservoirs both for irrigation and for power purposes, which I hope will be utilized to correct man's folly and prevent many disasters similar to those which have recently occurred in Indiana and Ohio. While the control of the Lower Mississippi by reservoirs is impracticable, there are numerous smaller streams where they can be used with excellent results.

It is questionable, however, whether such reservoirs should be built with the control of our rivers the first object of consideration. It will, to be sure, saddle the cost on the United States treasury, but to close down a power plant and stop the growth of crops every time the navigation of a minor stream is interfered with, I do not consider would be a wise proceeding.

I am also skeptical of government ownership. It may have worked satisfactorily in irrigation projects, but my own experience with government ownership of water power makes me suspicious. I have found that when the government buys water power, the power companies consider it worth $25 per horsepower per year, but when conditions are reversed, and an attempt is made to lease it, the price drops to ten cents.

Wherever it will pay to build a dam for power purposes, capital stands ready to construct it, if it can obtain the franchise. By regulating the franchise, the reservoir can also be used to restrain local floods.

The systematic conservation and regulation by the Government of a river from its source to its mouth sounds most attractive, suggesting a scientific solution of every problem of river hydraulics, but instead I greatly fear that it is the voice of a siren luring the people to an open pork barrel for every stream in the United States.

A Two-Company Field Work*

BY

Capt. F. B. WILBY
Corps of Engineers

GENERAL SITUATION.

The two-company work described in the following pages is a portion of a position about 2½ miles long, presumably held by one brigade of a division, having two brigades on the line, with one in reserve. This division being one of several occupying an extended defensive position.

A position sketch of the brigade front was made by the officers of the First Battalion of Engineers, and the different works necessary (fire trenches, support trenches, artillery positions, etc.) were located thereon, after an examination of the ground in the field.

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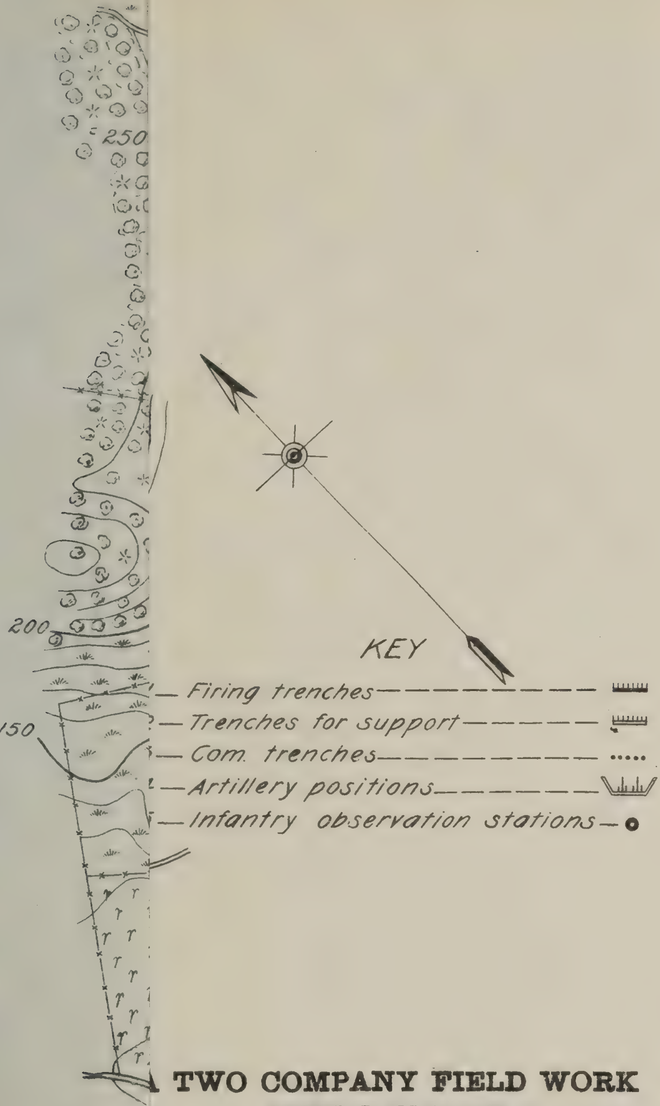
*The great amount of interest evinced by officers of the Corps of Engineers in the solution of the "Two-Company Field Work," by Captain Wilby, determined the editors to try to collect additional information. Accordingly, Plate A was obtained through the courtesy of the Geological Survey from advanced sheets of a new survey they are making of the vicinity in which the work is supposed to be located. Plate A (opposite page 448) at the end of Captain Wilby's article, has been added for the particular purpose of showing the general defensive line of which the half-brigade front treated of in Captain Wilby's article forms a part. The site of the two-company work is at the center of the circle marked P in upper center of plate. The defensive line is supposed to follow the ridge running generally east and west from the right border of the plate to about the site of the two-company field work, where it turns to the northwestward. The general situation is that of a field army occupying this line and its extension on a curve with a radius of 6 or 7 miles for about 15 miles, with both flanks resting on a river. The pictures were taken by Captain Wilby at points picked out on the ground by him and the editor.

The MEMOIRS will be glad to publish further discussions and comments on this article in succeeding numbers of the magazine. It is believed that the additional plate, together with the pictures, give sufficient information for a fairly complete solution of the problem, and the editors will be obliged to anyone who will submit such a solution.—ED.



A TWO COMPANY FIELD WORK

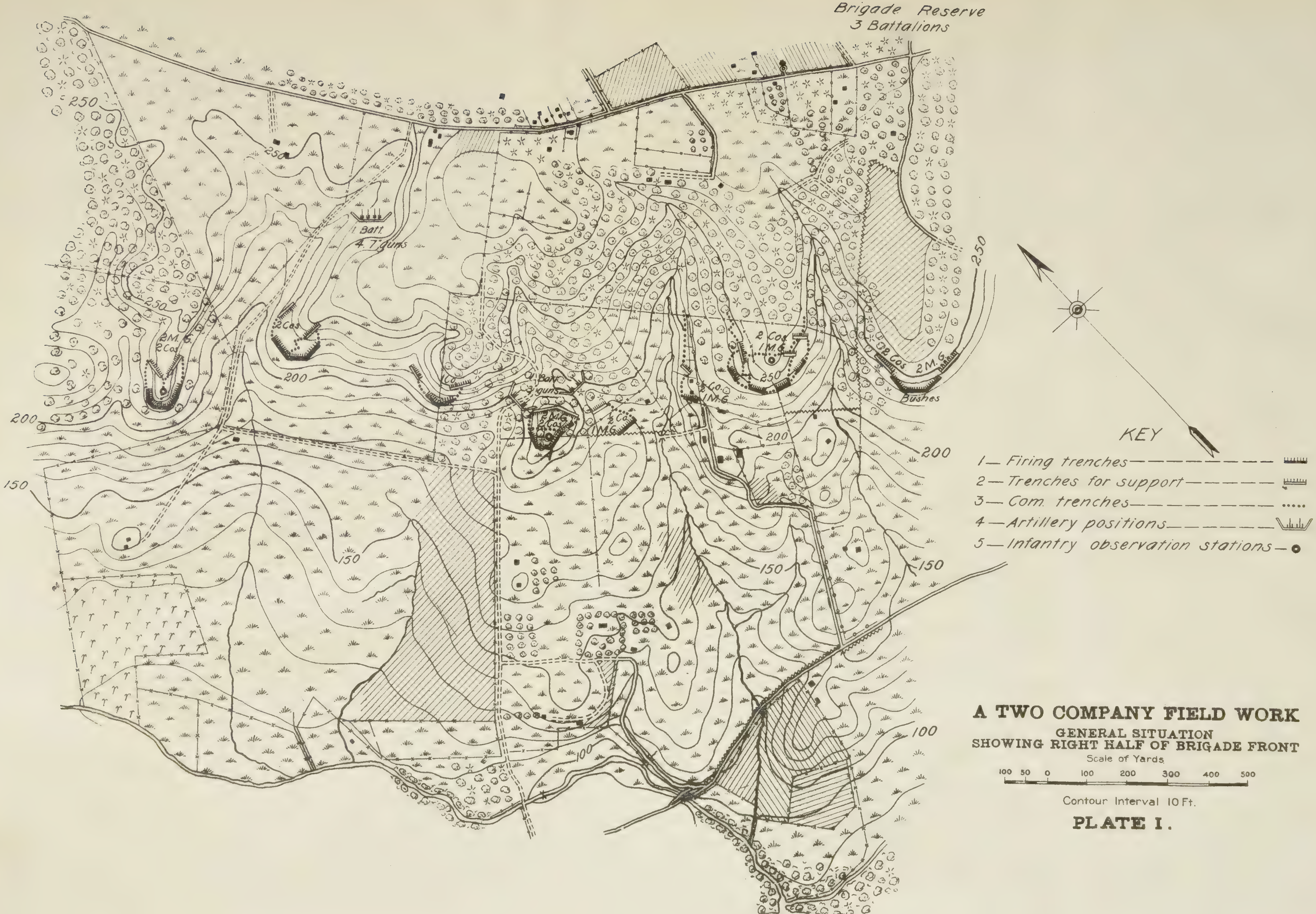
GENERAL SITUATION
HOWING RIGHT HALF OF BRIGADE FRONT

Scale of Yards

100 50 0 100 200 300 400 500

Contour Interval 10 Ft.

PLATE I.



A TWO COMPANY FIELD WORK

GENERAL SITUATION
SHOWING RIGHT HALF OF BRIGADE FRONT

Scale of Yards

100 50 0 100 200 300 400 500

Contour Interval 10 Ft.

PLATE I.

The *right half* of this line is shown in Plate I, and the two company work near the center of the line at A was selected to be worked up in detail, with large scale plan of site and all necessary working drawings, cross sections, etc.

SPECIAL SITUATION.

It is seen from Plate I that this two-company work is one of a battalion group disposed as follows: On the left, two half-company trenches with machine-gun emplacements, commanding the open slopes of a creek bottom; in the center the two-company work on a detached hill with open ground in front and wooded in rear; while the right is held by one company entrenched at the foot of a steep wooded slope. The whole group covers an almost ideal location for a 3-inch battery used as a "stabbing" battery, with two guns enfilading the front in each direction.

The work in the center, to be held by two companies and two machine guns, is the one selected for examination in detail.

It is the only position in this part of the line which lends itself in any particular to all-round defense, and though too restricted in area to accommodate a battalion, it will with the two adjacent companies form a strong supporting point, which should hold out if attacked in flank or rear for at least sufficient time for the active reserves to come to its assistance.

The special situation, then, which this work is to meet is an attack from the South, Southwest, and West, with a possible attack from the rear by any of the enemy who may obtain a temporary lodging on the hills to the North and East.

CHARACTER OF WORK.

The work is to be constructed by the troops after their arrival on the ground, and with the materials available on the site.

It is assumed that the time available is from *one to two days,* and that the enemy will attack with nothing heavier than 3-inch field guns. In case heavier artillery is used by the attack, the cover of course should be thicker, and the designs would differ materially, but for the purpose of this problem it was assumed that the work was to withstand 3-inch field guns only. It is believed that this type would be the first aimed at in any works constructed by the troops, and as time and material become available the cover would be strengthened.

GENERAL PLAN.

A large scale map of the site is shown in Plate II, with the plan

of the work located thereon. All trenches and positions were first staked out on the ground in the location selected, and then bearings and distances taken and the lines plotted on the map in their true position. The plan given is therefore not the solution of a map problem, but shows the result of lines selected on the ground. A vertical section of the hill on the line Y-Z is shown in Plate III. This section is without vertical exaggeration, so gives a good idea of the slopes, and the relative position of the firing trenches, cover for supports, obstacles, etc.

The plan, Plate II, shows the location of all necessary works for the defense, such as firing trenches, cover for supports, obstacles, observation stations, machine gun positions, communications, etc.; all of which will be described more in detail under the appropriate headings. In studying these plans the time and men available must be kept in mind. However desirable thicker overhead cover may be, or wider or deeper trenches, the work must be kept down to the minimum and our designs must not be of such magnitude that the work will be only half finished when the attack commences.

LOCATION AND LENGTH OF FIRING TRENCHES.

Our firing trenches must be selected so as to effectively cover the foreground to the South, Southwest, and West. Upon an examination of the ground it was seen that the following points must be considered in selecting this location:

a. The parapet must be as low as possible for purposes of concealment;

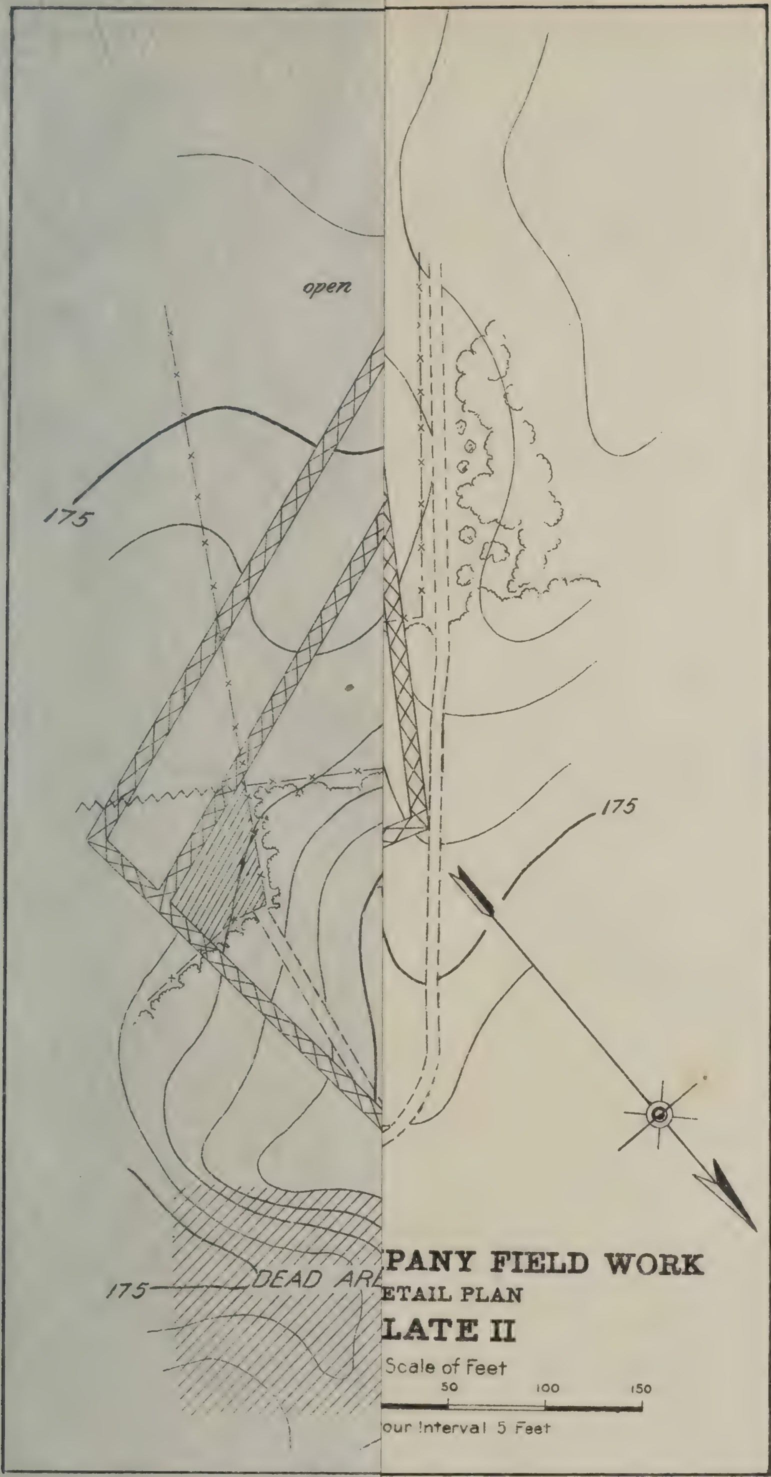
b. Due to this low parapet, the line must be well over the military crest to cover the immediate foreground;

c. The distance of the trench from the edge of the woods must be great enough to avoid undue losses from shots aimed at the edge of the woods;

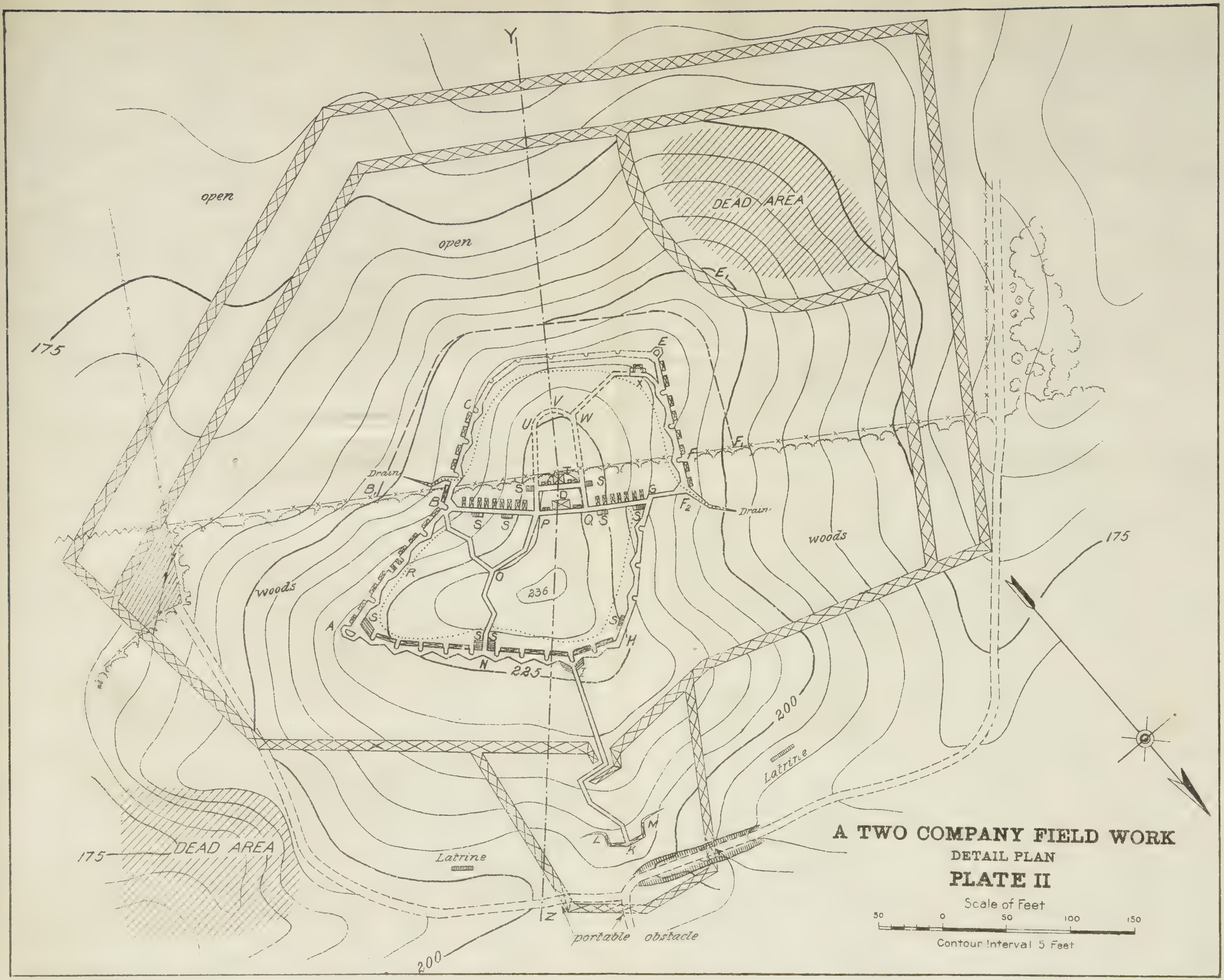
d. The foreground is open, but rolling, so that a trench at the foot of the slope is not feasible, as it would not have a grazing fire and would leave many dead areas. The trench must therefore be located at such an elevation as to command the majority of these areas;

e. The flank trenches should slope well from front to rear, not only for purposes of proper drainage, but principally to avoid visibility.

Points *c* and *d* above apparently conflict, and a compromise be-



PANY FIELD WORK
ETAIL PLAN
LATE II



A TWO COMPANY FIELD WORK

DETAIL PLAN

PLATE II

Scale of Feet

50 0 50 100 150

Contour Interval 5 Feet

tween the two must be accepted. After a consideration of all of the above points the line BCEF, Plate II, was tentatively decided upon and staked out. Upon measuring the length of this line, it was found to be about 110 yards in length.

Fig. 1 (upper). General view of the hill on which it is proposed to locate the two-company redoubt. The hill is the bald one in the upper right center of the picture, distant about 1,600 yards from camera set up at point X, Plate A, across the valley nearly southwest from the hill.

Fig. 2 (lower). Same hill on extreme upper right-hand edge of picture, taken so as to show the rolling nature of the foreground as it appears from the ridge at Z, Plate A, on the unimproved road about 750 yards southeast of the hill.

The question as to the proper length of firing trench for two companies depends upon the strength of the companies, and the disposition of the troops during the attack.

The infantry company as given in the recent "Report on the Organization of the Land Forces of the United States" (Chap. VI, p. 38) consists of sixteen squads or 128 rifles. If half of these are held in support, that gives eight squads from *each company* on the firing line, or for two companies sixteen squads on the firing line, with an equal number in support. Now, if we prepare firing trenches long enough to accommodate these sixteen squads with one man per yard, we will have a density of not exceeding two men per yard when the entire support has reenforced the firing line. This appears rather crowded, but is not impossible if necessary; however, such a dense firing line would seldom be attained in practice, for the following reasons:

a. The company would seldom go into combat mustering its full strength; and it is thought twelve squads would be nearer the average available number of rifles;

b. Casualties would deplete the firing line before the entire support was absorbed;

c. Even up to the last, a certain portion of the support would be held out for use in delivering a counter-attack.

Due to the above causes, the density of fire with the disposition assumed above would probably never exceed 1½ men per yard (or 2 feet per man). This is the maximum density prescribed by German "Regulations for Field Fortifications, 1906," par. 39, and is sufficient to deliver the maximum fire effect without overcrowding the men.

We may therefore determine the length of our firing trench to be such as to give *1 yard of firing crest per man*, to *half* of the *authorized* strength of the garrison in *rifles.* This, for two companies of 128 rifles each, gives 128 yards; but to this must be added 1 yard for each traverse, and sufficient space for the necessary machine-gun positions and observation stations. To accommodate sixteen squads with one traverse between every two squads, this would increase the necessary length of trace to 128+15=143 yards, without considering machine guns and observation stations.

Our line as staked out measures only 110 yards, and to find the additional length necessary (at least 33 yards) we may do one of two things:

a. Lengthen the line by moving down the slope farther;

b. Extend one or both flanks to the rear.

Fig. 3. Panoramic view taken from about the center line of the front trench, as proposed by Captain Wilby, showing the field of fire from the front trench, with the exception of that toward the left flank which is shown in Fig. 4. Fig. 1 was taken from the bare hill very nearly the center of Fig. 3, just over the left edge of the row of trees to the right of the barn in the immediate foreground. The stake in the ground alongside of the man sitting down near the edge of the picture, lower left-hand side, is on the line of the trench as proposed by Major Hart, Major Connor, and some others.

If the line was lengthened by the first method it would take some such position as shown by the broken line B_1F_1, Plate II, with the firing crest lower by at least 10 feet vertically. After an inspection of the field of fire from this lower position, it was abandoned owing to the large areas in the immediate foreground not under fire from this location.

The second method must then be used, and the flanks extended to the rear. This additional length can easily be secured on the left from B to A, where we have 40 yards of trench well located for fire on an attack from either the left front or left flank; while on the right we will only add one more squad (or 9 yards) from F to F_2 in order to strengthen the fire from this flank.

The firing trench then as finally located is $ABCEFF_2$, and is apparently much stronger on the left flank than on the right. This apparent inequality is, however, offset by the fact that the foreground on the right front is open and of an even slope, affording an excellent field of fire up to the long ranges, and therefore giving every advantage to the defense; while, on the other hand, the foreground on the left front is broken and rolling, being cut up by ravines and gullies affording cover for the attack even at short ranges (Plate I). The firing trench is therefore so located as to concentrate the greatest fire on the most vulnerable flank.

One other point should be mentioned, and that is the "dead area" indicated on the right front (see Plate II). This area can only be commanded from the point E_1 and if the salient at E is extended to include E_1 it would have the following disadvantages:

a. It would face the front trench too much to the left;

b. It would face the right flank trench too much to the rear;

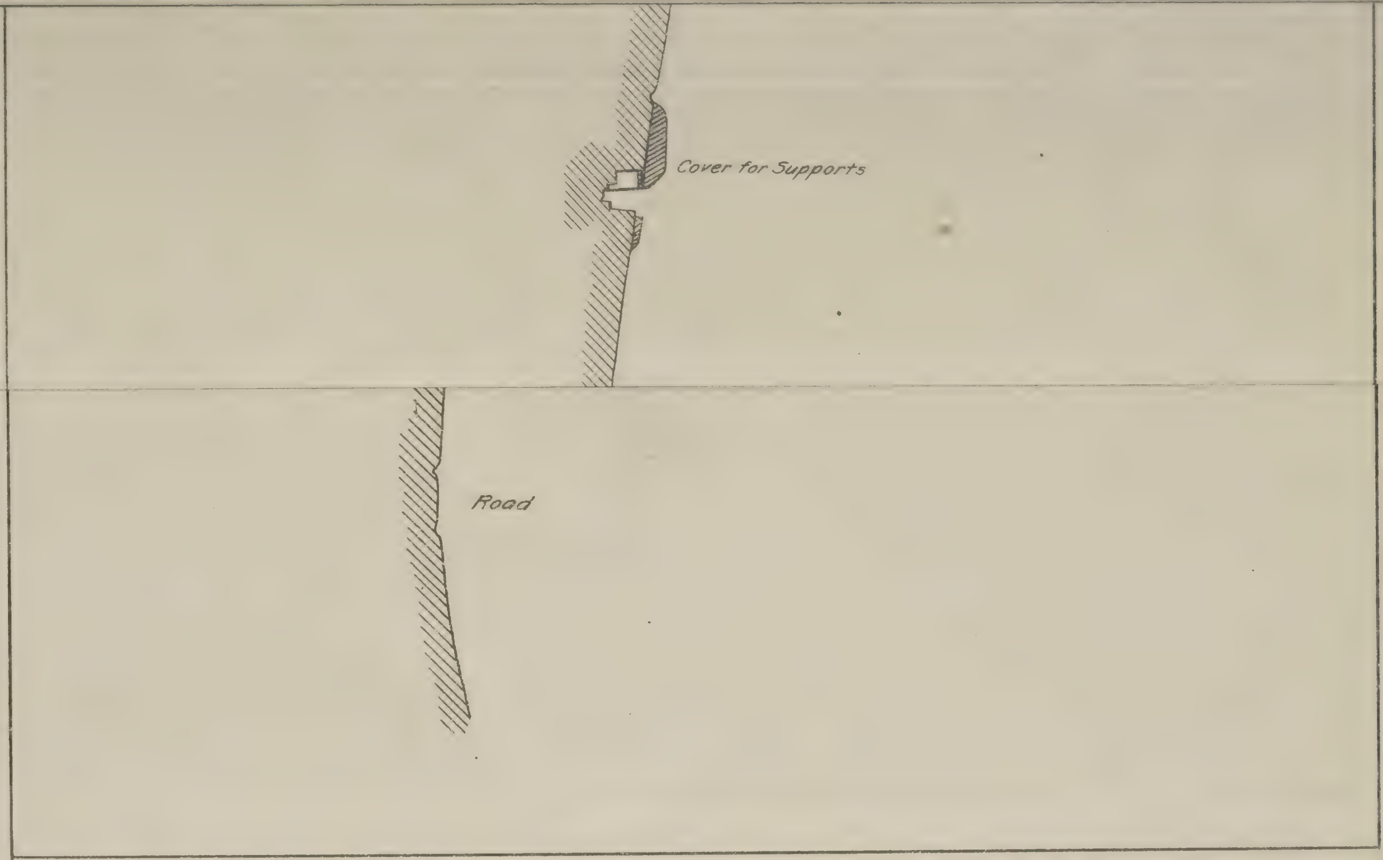
c. It would prevent fire from either the front or right flank trenches from covering an attack on the right front;

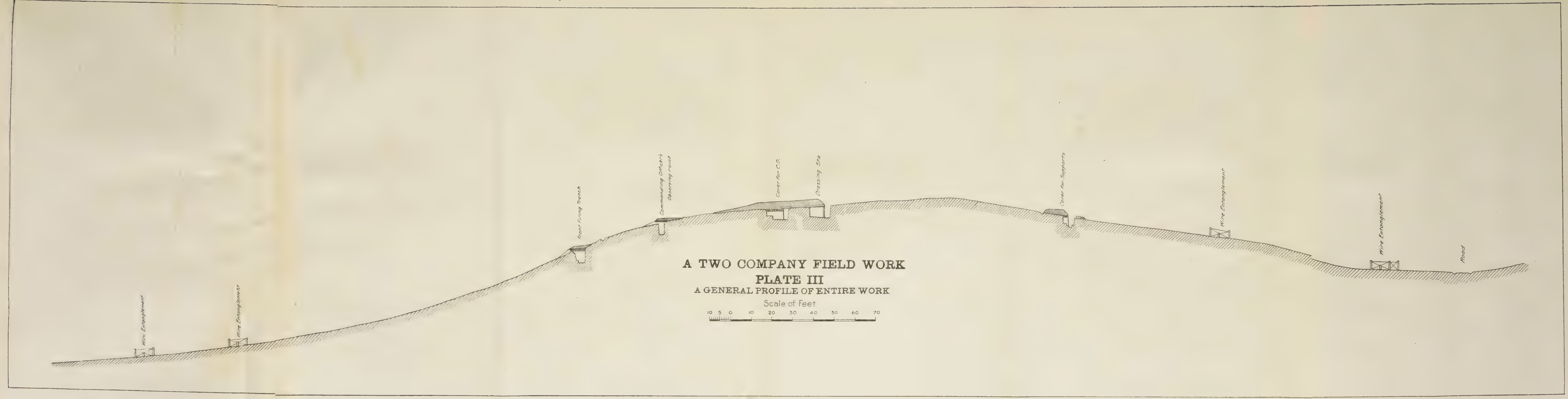
d. It would present a weak salient to the enemy.

It was therefore decided to hold the trench in the retired position, leaving the dead area as shown but protecting the salient by deflecting obstacles beyond this area, and by an additional interior obstacle which would prevent any of the enemy who might have obtained lodgment there from rushing the work. A machine gun located at E gives added protection with its high rate of fire, besides enfilading any attack on either the front or right flank.

LOCATION OF SUPPORT TRENCHES.

In the preceding paragraphs we have said that during the early


Cover for Supports
Road



A TWO COMPANY FIELD WORK

PLATE III

A GENERAL PROFILE OF ENTIRE WORK

Scale of Feet

10 5 0 10 20 30 40 50 60 70

stages of the attack about half the garrison will be under cover in rear, acting as a support. The location of cover trenches, of course, depends on the location of the firing trenches, but in deciding on the position of the latter the probable location of the former must constantly be kept in mind.

The support trenches are preferably located out of sight of the enemy, on the reverse slope of the hill to minimize the digging for the necessary cover; not closer than 50 yards in rear, to avoid catching the overs aimed at the firing trench; and not much over 100 yards away in order to be within prompt supporting distance. Such a location is shown on Plate II at ANH, which is 80 yards in rear of the front trench, and on the reverse slope. One other location which might be considered is 30 yards farther to the rear, or 110 yards in rear of the front trench, along the "military crest" of the reverse slopes, so as to command the otherwise "dead area" along the road. This latter position has its disadvantages as follows:

a. It brings the support trench nearer and under the commanding hills to the North, in case of an attack from that direction;

b. It places the supports at too great a distance from the fire trenches;

c. Even if located thus so as to command certain areas along the road it will then still leave a considerable dead area within 30 yards in the gully at the left rear, marked *Dead Area* (see Plate II).

It was therefore decided to locate the support trenches as shown at ANH with a clear field of fire for at least 30 yards, covering the obstacle but not the gully beyond containing the road; this gully to be flanked by one squad in a covered trench LKM, surrounded by an additional obstacle.

The trench ANH is about 65 yards in length and would furnish cover for about nine squads of the support. As cover is required for about half the garrison, or sixteen squads, we must arrange cover for seven more squads elsewhere.

This is conveniently located in the trench BPQG, parallel and midway between the front firing trenches and the support trench. This trench is required for purposes of communication and, as it is sheltered from view by the woods, may well be utilized to furnish the additional cover for the supports.

PROFILES OF FIRING TRENCHES.

In general, the profile adopted for the main firing trenches is about as shown in Fig. 1, Plate IV, being an open trench of low

parapet for purposes of concealment. The trench is to be traversed every 9 yards, as shown in plan (Fig. 2, Plate IV); with two splinter-proof shelters under the parapet between every two traverses, each shelter to accommodate four men; thus one squad of eight men is the normal garrison between traverses. This standard design was not departed from except for important reasons in each case.

The advantages of an open trench without overhead cover or loopholes for a *field work* is recognized by the German "Regulations for Field Fortifications," which mentions loopholes only under siege operations (paragraphs 131 and 132). The reason for this is evident, in that siege operations are conducted at *short ranges* where concealment of a position is no longer possible, and where cover from the enemy's fire is the main consideration. In a work of the type under discussion, where concealment of the general position is of primary importance, overhead cover or loopholes should never be added if by so doing the general position is made visible from long ranges.

There is no question as to the relative visibility of an open trench or one with loopholes when located on level or nearly level ground; the advantage is all with the former. But as the ground slopes more and more to the front, the open trench becomes more and more difficult to conceal, due to the exposed earth surface in rear, which was formerly concealed by the parapet. This limit of slope, above which it is very difficult to conceal the open trench, is about 17 to 18 degrees; so for slopes steeper than this the disadvantages of the loopholes have disappeared, and they may be installed without sacrificing invisibility. If the head cover in these cases is carried to the rear over the entire trench, it has the added advantage of protecting the men in the trench from splinters of shells exploding on the slope above.

For the above reasons the front firing trench from C to E, which occupies a slope varying from 19 to 23 degrees, is designed with overhead cover as shown in Fig. 3, Plate IV. This overhead cover then takes the place of the splinter-proof shelter under the parapet, which is omitted, and the bottom width of trench is widened to 2 feet 9 inches in order to permit passing in rear of the line of men sitting on the firing step. The traverses in this section are all made very short, only 2 feet in length, to avoid the deep excavation necessary (of at least 10 feet) if passage-ways are dug around a full-length traverse.

Section thr

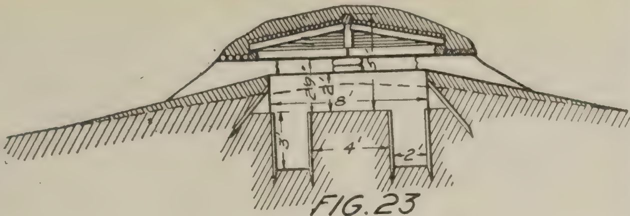


FIG. 23
Section 3-4, Fig. 21

to P

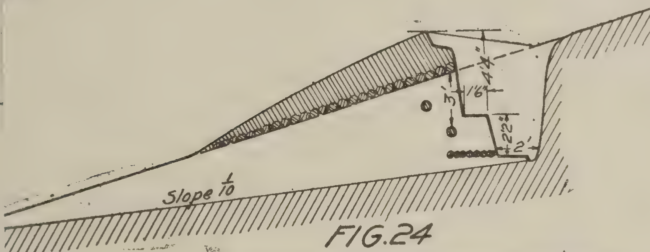
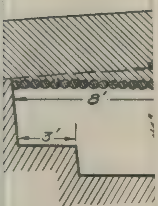


FIG. 10
Station at T
at D (with roof

FIG. 24
Longitudinal section through drainage ditch at B (showing location of latrine)



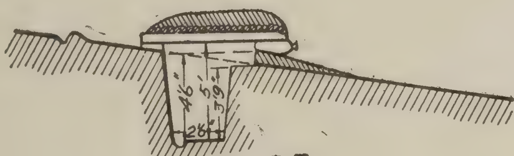
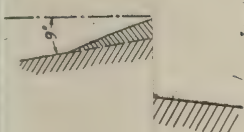


FIG. 25
Profile of Firing Trench from L to M



A TWO COMPANY FIELD WORK
PROFILES AND PLANS
PLATE IV

Scale of Feet

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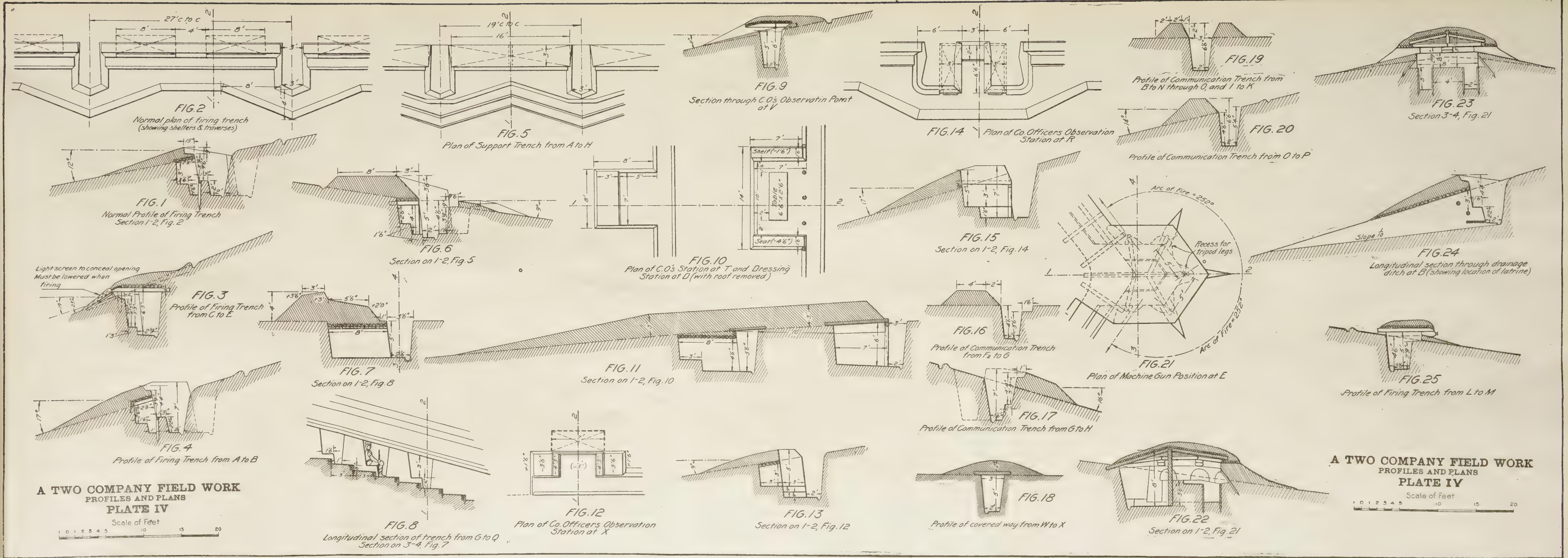


FIG. 1 Normal Profile of Firing Trench Section 1-2, Fig. 2

FIG. 2 Normal plan of firing trench (showing shelters & traverses)

FIG. 3 Profile of Firing Trench from C to E

FIG. 4 Profile of Firing Trench from A to B

FIG. 5 Plan of Support Trench from A to H

FIG. 6 Section on 1-2, Fig. 5

FIG. 7 Section on 1-2, Fig. 8

FIG. 8 Longitudinal section of trench from G to Q Section on 3-4, Fig. 7

FIG. 9 Section through C.O.'s Observatin Point at V

FIG. 10 Plan of C.O.'s Station at T and Dressing Station at D (with roof removed)

FIG. 11 Section on 1-2, Fig. 10

FIG. 12 Plan of Co. Officers Observation Station at X

FIG. 13 Section on 1-2, Fig. 12

FIG. 14 Plan of Co. Officers Observation Station at R

FIG. 15 Section on 1-2, Fig. 14

FIG. 16 Profile of Communication Trench from F_2 to G

FIG. 17 Profile of Communication Trench from G to H

FIG. 18 Profile of covered way from W to X

FIG. 19 Profile of Communication Trench from B to N through O, and I to K

FIG. 20 Profile of Communication Trench from O to P

FIG. 21 Plan of Machine Gun Position at E

FIG. 22 Section on 1-2, Fig. 21

FIG. 23 Section 3-4, Fig. 21

FIG. 24 Longitudinal section through drainage ditch at B (showing location of latrine)

FIG. 25 Profile of Firing Trench from L to M

A TWO COMPANY FIELD WORK
PROFILES AND PLANS
PLATE IV
Scale of Feet

A TWO COMPANY FIELD WORK
PROFILES AND PLANS
PLATE IV
Scale of Feet

Fig. 4 (upper). View of the field of fire taken from the same point in the front trench as Fig. 3 and extending that view to the left. This also gives a view of the folds in the ground to the left of the field of fire. See, also, Fig. 2 of flank views of same folds.

Fig. 5 (lower). View of same sector of field of fire as shown in Fig. 4, but taken from the trench proposed by Major Harts and others at the point indicated by the stake in Fig. 3. Figs. 4 and 5 show the increase in the amount of cover afforded attackers coming from the direction of the barn near the center of the picture if the lower trench proposed by Major Harts is used instead of the upper trench proposed by Captain Wilby.

The firing trench from B to C, and E to F_2, is of the standard profile (Fig. 1, Plate IV), as the slope is only 13½ and 12 degrees, respectively.

From A to B the trench is masked by woods, and the parapet may be as high as desired; a 2-foot parapet with the area of cross section equal to that of the trench is the most economical of labor, so has been adopted (see Fig. 4, Plate IV).

In all profiles of firing trenches, the elevation of the firing crest above the firing step varies with the slope of the ground. On level ground, from a series of exhaustive experiments made by the writer, the most convenient height for the average man was determined as 4 feet 7½ inches; this height may be separated into 3 feet 9.3 inches from the firing step to the elbow rest, and 10.2 inches from the elbow rest to the firing crest. The width of the elbow rest arrived at from the same experiments was 15.6 inches.

However, it is a well-known fact that this height (from the firing step to the firing crest) must be decreased when firing down a slope; for steep slopes some authorities give the proper height of parapet as 4 feet. The necessary deduction in each case can only be found by experiment. It is thought that a good general rule is to deduct 1 inch from the height of 4 feet 7.5 inches for every *5 degrees* of slope, but in deducting this amount care must be taken that the height of 3 feet 9 inches from the firing step to the elbow rest is retained in all cases, making all the reduction in the height from elbow rest to firing crest.

PROFILES OF SUPPORT TRENCHES.

Shelter in the support trenches is furnished for at least one-half of the authorized strength in rifles. This shelter is to consist of seats with overhead cover, proof against 3-inch shell. Larger covered shelters, furnishing sleeping quarters for the entire garrison, are considered unnecessary, inasmuch as the command will be bivouacked on the wooded reverse slopes of the hill. During any attack by the enemy it is assumed that all men will be awake and occupying either the firing trenches or the shelter for supports.

The support trench between A and H (shown in plan and section in Figs. 5 and 6, Plate IV) furnishes cover for nine squads, one squad between every two traverses except at N where counter-attack steps are located. This trench is arranged to fire to the rear with a field of fire of about 30 yards, up to and including the obstacle (see Plate II).

Sufficient cover for eight more squads is furnished in the trenches

Fig. 6 (upper). View from top of hill where it is proposed to put the commanding officer's look-out station, looking to the left with the line of woods shown in Plate I just off the left edge of the illustration. Fig. 2 was taken from the road running in front of the house and fields in the upper center of the picture where that road crosses the grassy knoll near the right-hand

edge of the picture. The ravine (on either side of which Captain Wilby has proposed to put a half-company work) is the one running down through the center of this illustration.

Fig. 7 (lower). View of foreground looking directly to the right from look-out station, showing fields of fire for works to the right of the two-company work. In all of these views the character of the hills across the valley from which the attack would come is shown.

from Q to G and from B to P (cross sections shown in Figs. 7 and 8, Plate IV). As these trenches are located normal to the contours the bottoms are stepped and the shelters are placed parallel to the contour and perpendicular to the trench. The earth cover over these shelters is so sloped as to furnish a parapet to be manned by the support in case the front trench is carried.

TRAVERSES.

All trenches arranged for fire in any direction are traversed. (See Plate II.) These traverses are 1 yard thick at the surface of the ground and as long as the trench is wide, except on the front firing trench as noted above.

In trenches BC and EF_2, the tops of the traverses are as high as the firing crest in front, sloping to the rear, in order that they may be hidden; but in trenches AB and GH, which are well masked by woods the traverses can be made higher than the firing crest, thus giving additional protection against enfilade fire, which is needed as both trenches slope down hill toward the enemy.

All passageways around traverses are made of such dimensions that a litter can be carried around without difficulty. (See plans in Figs. 2 and 5, Plate IV.)

OBSERVATION STATIONS.

The commanding officer's observation station must be so located as to command a view of the foreground to the front and both flanks. Such a position is difficult to obtain without erecting a station of such a height that it would serve as a prominent target for the enemy's artillery. The best solution of this difficulty is to locate three observing points of low command (U, V, and W, Plate II), connected by a covered way so that the commanding officer can pass quickly from one to another. Even such points, however, are on a prominent crest and all openings must be concealed by screens when not actually in use. (Profile shown in Fig. 9, Plate IV.)

Shelter for the commanding officer's station (proof against 3-inch shell) is located at T under the shelter of the woods, and is to be built only if time permits. (Plan and cross section shown in Figs. 10 and 11, Plate IV.) Two shelters for either orderlies or telephone operator are furnished, one on either side of the C. O. station. (See Plate II, no detail drawing shown.)

Two types of company officer's observation stations are shown, to

be built if time permits, one at X for the right company observing to the front and right flank, and one at R for the left company observing the left front and left flank. The one at X was drawn back of the trench, as there was no point in the trench, except at E, where the front of both faces could be seen. The one at R, though not placing the company commander in the center of his command, is at much the best location so far as the view to the front and flank is concerned. (Figs. 12, 13, 14, and 15, Plate IV, show plans and profiles of these observation stations.)

DRESSING STATIONS.

A dressing station is located centrally at D with cover against 3-inch shell for an operating table, and with sheltered seats on either side for those who may be waiting; these shelters to be built only if time permits; plan and profile shown in Figs. 10 and 11, Plate IV.

COMMUNICATIONS.

Communicating trenches F_2G and GH, are designed for fire to the flank and rear. The covered way WX must be built simultaneously with the observation trench UVW, not only for purposes of communication, but also for the proper drainage of that trench. Trenches BON, PO, and IK, will not be started until the rest of the work is finished. (For profiles, see Figs. 16, 17, 18, 19, and 20, Plate IV.)

MACHINE GUN POSITIONS.

The machine guns are located at A and E, Plate II. The first covers with its fire an arc of almost 280 degrees, commanding the left front around far enough to include the obstacle in rear up to the entrance. The other at E has an arc of fire of about 252 degrees, enfilading an attack on either the front or right flank of the work. Plan and profiles of this position at E are shown in Figs. 21, 22, and 23, Plate IV, designed to accommodate the automatic machine gun on the tripod mount, as at present in use by our army.

The design of a machine-gun position with overhead cover, which is not visible for miles around, is rather difficult, and especially so if the guns must be prepared to shoot down a steep slope. The openings or firing slits between crest of parapet and roof are 9 inches wide; these slits must be effectively screened, when not in use, by hinged screens of light sticks covered with long grass,

leaves, or bagging of such a color as to blend with the surrounding slopes, and prevent the casting of any heavy shadows visible from a distance.

The position at A is not shown in detail, as it would be similar to that at E except that as it is masked by the woods, its concealment is not of such importance.

DRAINAGE.

The trenches have all been laid out with a view to proper drainage. The high points are at E, D, I, and midway between O and N. From these four points the water in the trenches flows both ways, passing through gutters (shown in all profiles) to the main outlets near B, and at F_2; except for the water in trench IK, which flows out at L and M. Surface ditches are placed on the slope *above* all trenches to catch the surface water, and prevent it from filling the trenches. These ditches, shown on Plate II with *dotted lines,* conduct the surface water to points where it can be quickly carried off through the outlets at B and F_2.

LATRINES.

The latrines for use at all times except when actually under fire will be located well to the rear outside of the obstacle, probably at some such position as marked on Plate II.

For emergency use during action two latrines are located under cover, one in each drainage ditch at B and F_2. These may take the form of two poles as shown in Fig. 24, Plate IV, which would only be used in case of temporary occupancy of the work, or they may be dry earth closets which would be removed and emptied at night.

OBSTACLES.

A wire entanglement is to be located as shown on Plate II; continuous around the entire work, at all points under fire from the trenches, being drawn in for this purpose to a distance of 30 yards in the rear, while its average distance from the firing trenches on the front and flanks is about 65 yards. It should be constructed in two parallel lines about 10 yards apart, except in the rear. One line, of course, being constructed first, and the other line added if time and material were available.

The construction of the entanglement, as shown in Plate II, requires a great deal of labor and material. It totals about 3,800 square yards, which with 30 feet of wire per yard, would require

Fig. 8 (upper). Field of fire of half-company work on the left of the ravine shown in Fig. 9. It will be noted here that the slopes in the vicinity of the barn and on up the ravine to this work are well covered by it. The trees in the foreground would, of course, be cut down.

Fig. 9 (lower). View taken from the location of the half-company work on the side of the ravine next to the two-company work looking along the field of fire of that work. This picture shows in the center the road from which Fig. 2 was taken where it crosses the grassy knoll.

114,000 feet of barbed wire. Assuming that one man can construct 2.75 square yards of entanglement per hour, it would take *seventy men* working for *two days* of *ten hours* each, to construct the 3,800 square yards shown. This labor and material can be reduced one-half if only a single line is constructed and the extra obstacles around the entrance in rear and inside the dead area at E are omitted.

If material is not available for construction of wire entanglements as shown, the obstacle will take the form of an abatis with the addition of such barbed wire as may be obtained from nearby fences.

CLEARING.

The woods between the trenches BA, AH, HG, GF_2, F_2F, and the obstacle in their respective fronts (see Plate II), will be thinned out to such an extent that fire from the trenches will command the obstacle. However, a fringe of large trees will be left along the edge of the woods, in order to mask the trenches in rear without interfering with fire from these trenches to the front. Little, if any, clearing will be done inside the work.

MISCELLANEOUS.

Steps must be provided for entering the work at I, and numerous others for the purpose of possible counter-attacks are shown in Plate II, and marked S.

The area in rear of the work along the road, which is not covered by fire from the gorge trench ANH, may be swept by fire from the trench LKM arranged for one squad with overhead cover as shown in Fig. 14, Plate IV. This trench is considered of minor importance, and would not be constructed until the remainder of the work is finished.

ESTIMATE OF TIME FOR CONSTRUCTION.

The field work described above is designed with a view to completion by the troops in two days of ten hours each. The important elements of the defense, including all firing trenches, cover for supports, positions for machine guns, and a single thin line of obstacle, could probably be completed the first day.

A rough estimate of time and working parties for the first day's work is given on the next page.

Trench.	Working party.	Total excavation.	Cubic feet.	Time.
		In cubic feet.	*Per squad.*	
AB	4 squads	3,152	788	9 hrs. 51 min.
BC	3 squads	2,469	823	10 hrs. 17 min.
CE	6 squads	4,668	778	9 hrs. 43 min.
EF_2	4 squads	3,292	823	10 hrs. 17 min.
GH	2 squads	1,750	875	10 hrs. 56 min.
HA	7 squads	5,390	770	9 hrs. 37 min.
BP+½ PQ	3 squads	2,484	828	10 hrs. 21 min.
½ PQ+QG+GF_2	3 squads	2,331	777	9 hrs. 43 min.

Total 32 squads, *or two companies* for approximately *ten hours*.

NOTE: The position for machine guns would be prepared at A and E by the detachments serving them, and should be completed at least as soon as the trenches given above.

The time to complete the excavation above is based on *20 cubic feet per man per hour*, with only four men out of each squad working, that being the number of tools available. This leaves the other four men from each squad, or 128 men, to work on the obstacle, and to prepare material for overhead cover.

These 128 men could complete a single line of wire entanglement, 3 yards wide, around the entire work, a distance of 633 yards, in *six hours*; assuming that one man could construct 2.75 square yards per hour. The other four hours would be spent by the members of each squad in obtaining and placing timber in the overhead cover necessary in the trench excavated by that squad.

Of course, the men in each squad would be shifted about, every two hours, from the digging to the construction of the wire entanglement, and *vice versa,* in order that no one man would be kept at excavation for ten hours continuously. In this way it is thought that the average of 20 cubic feet per man per hour, the generally accepted rate in medium soil, could be kept up as estimated above and the work placed in a position of defense after approximately ten hours work, or at least in one day.

The auxiliary works needed, such as observation stations, commanding officer's station, dressing stations, latrines, additional communications, surface drainage ditches, additional obstacles and flank defense for dead area in rear could all easily be completed the second day.

CONCLUSION.

This two company field work is not so elaborate as the type which would be constructed in provisional or *strategical fortifi-*

cations constructed by civilian labor with two or three weeks time at their disposal; and yet it is much more elaborate and proportionally stronger than the simple standing trench thrown up almost in the presence of the enemy. In fact, it is intended as an example of the *tactical fortification* to be constructed by troops when preparing an extended front for immediate defense. The defense to be stubborn and possibly developing into the nature of a siege, and as the attack progresses the works to be strengthened and cover increased accordingly.

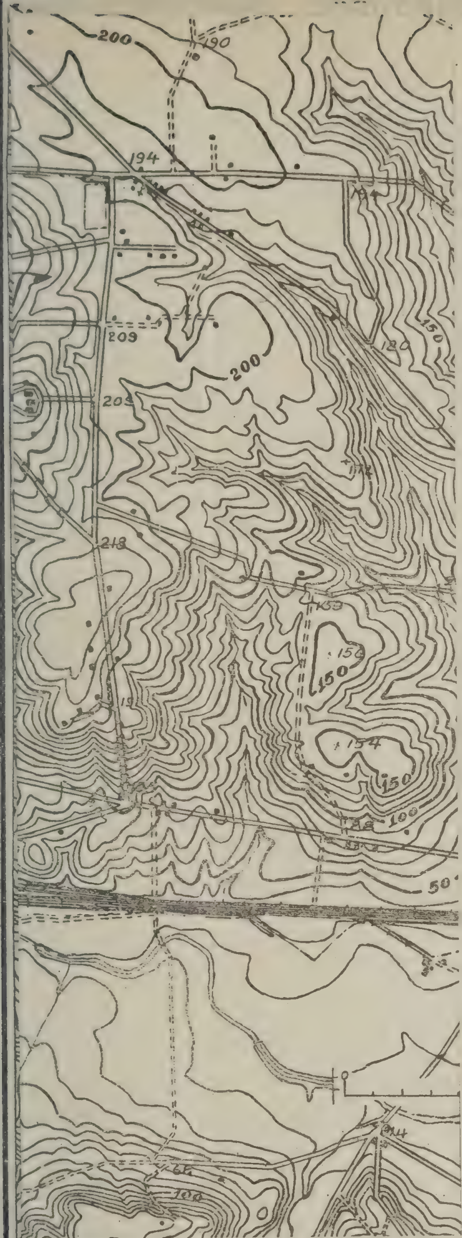
The shape of the work and relative disposition of the trenches can not in any way be considered typical. It is simply an attempt to illustrate the application of modern principles of fortification to a particular problem—that is, the preparation for defense of this particular piece of ground, remembering that it is to be only a unit in a general defensive line.

Discussion

Maj. McDONOUGH *and* Capts. BOND *and* SPALDING
Corps of Engineers

The discussion of a practical problem from the map alone is apt to be rather unsatisfactory, especially when, as in this case, there are discrepancies in the map. With reference to the slope in front of the work the writer gives three graphical presentations, no two of which agree, and all of which are at variance with the statement made in the text. In Plate I the front slope is roughly 8 per cent or 4½ degrees, in Plate II the same slope is shown as 24 per cent or 14 degrees, in Plate III it is 19 per cent or about 11 degrees, and in the text it is stated to be from 19 to 23 degrees.

The proper disposition of troops and organization of a defensive position depend very largely on the general strategical and tactical situation. In this article the general situation is not clearly stated. The mission of the force occupying the position is not clear, nor the general plan of the commander. The strength and composition of the force occupying the entire position is not given, although it is evidently at least a field army. The causes leading to the occupation of the position are not given, nor the strength and position of the enemy. There is no map showing the general nature of the terrain, the location of the important strategical points, communications, supporting troops, etc. It is not known which army is on the strategical defensive, nor whether the defense of the position under discussion is to be active or passive. There is no indication of the purpose of this defense, nor of the relation it bears to the general situation, whatever the latter may be. Engineer officers have been repeatedly, and not always unjustly, criticised for organ-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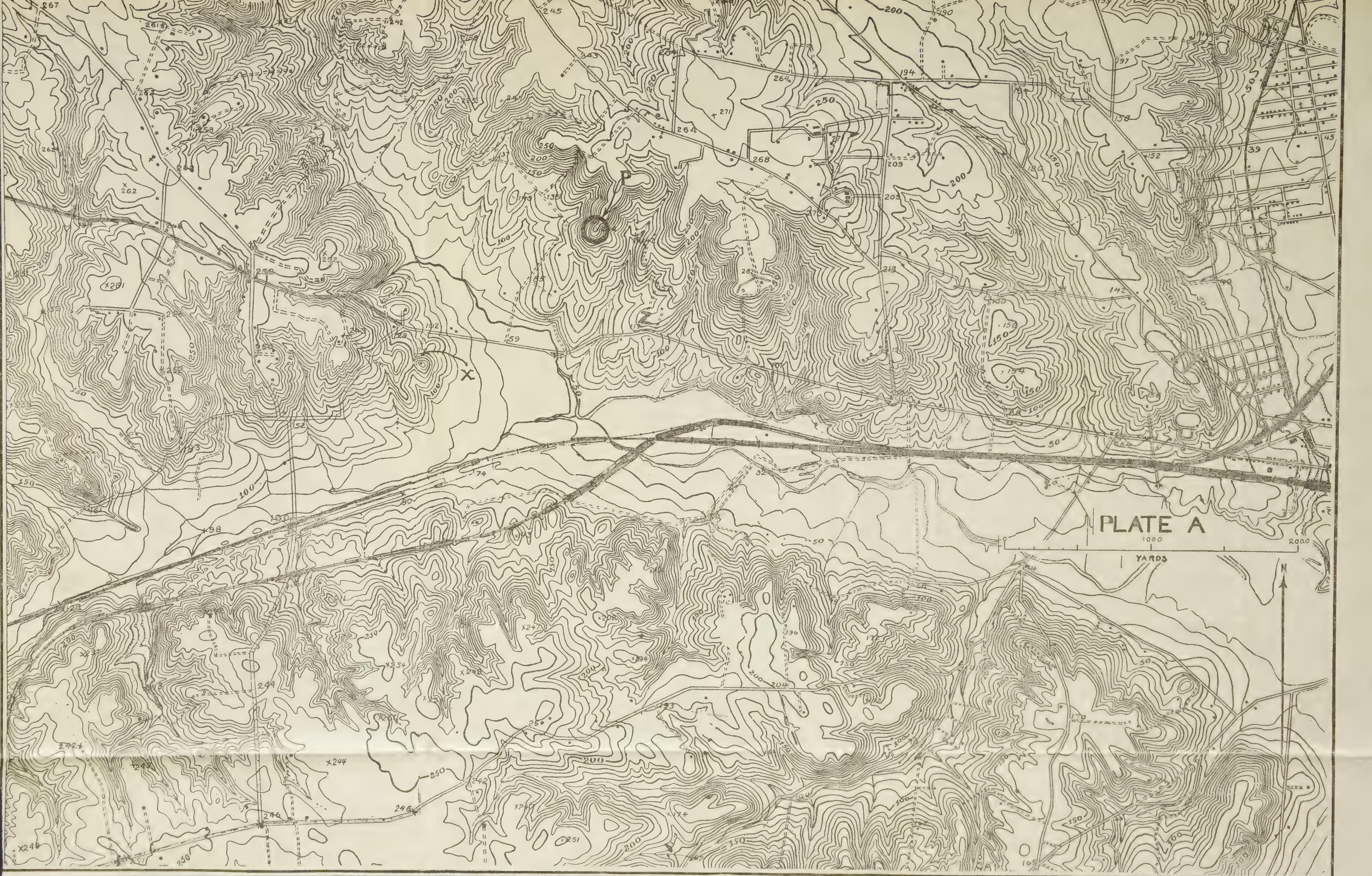
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izing defenses without due regard to the strategical and tactical requirements of the situation. Regulations and tactical writings are filled with warnings against this mistake, and history records many great military disasters which have resulted therefrom—in our own civil war, for instance, the Confederate disasters at Donelson and Vicksburg. It is an axiom of the science of war that defenses designed without due regard to the strategical and tactical situation will usually fail to accomplish their purpose.

This uncertainty as to the general situation makes it difficult to intelligently criticise details. Not knowing the situation, we can not see the occasion for the all-around defense of a small interior sector of a general line of battle, defended by field works. An attack from the rear is almost inconceivable in such a situation, and if the adjacent portions of the line have been captured by the enemy it is doubtful if any useful purpose can be served by two companies attempting to hold out after the loss of the general engagement. In a modern battle of any magnitude the attack will ordinarily come from the front, frontal resistance being opposed even to the enveloping attack. In fact, the position should be selected so that this method of attack will be the only one open to the enemy. If a defensive line is open to attack from front, rear, and both flanks, as well as from interior portions of the line itself, it is so faultily located that it would be better to seek a more favorable position. The position should be such that the enemy will be forced to attack its front, and it is to the development of frontal fire that efforts should be directed.

The disposition and proposed use of troops and artillery on this line is not in accordance with American organization and tactics, as follows:

Companies hold out supports, instead of being placed complete on the firing line;

Battalions have no battalion supports, thus the battalion commanders from the outset are largely deprived of any power to influence the course of action;

Regiments have no reserves, thus the regimental commanders from the beginning of the action have very little influence on its course. By this practice two of the regiments of the brigade are immobilized from the outset, and taken out of the hands of their commanders. It is by the use of his reserves chiefly that a commander can influence the course of an action. Here, the majors have no supports and the colonels no reserves;

The flanks of the brigade have no troops capable of active defense within 1¼ miles, in spite of the fact that the total trench development of the regiment shown (about 800 yards plus traverses), a large portion of which is not capable of effective frontal fire, could be adequately defended by two battalions, each having one company in support, thus leaving the regimental commander a suitable reserve of one battalion, and immobilizing at the outset on the brigade front but four battalions instead of six.

In addition to the immobilization of troops it is proposed also to immobilize a large number of machine guns. We regard this as a serious mistake. On this point Colonel Balck says (Tactics, Vol. I): "In employing machine guns in defense it should be borne in mind that they are unsuited for carrying on a protracted fire fight, and that the mobility of the machine guns can not be utilized when, from the very start they are assigned a sector to defend. In general, it will be advisable in defense to keep the machine guns at first with the reserve, and to employ them when necessary to reinforce the defensive line at threatened points, to prevent envelopment, to repulse an assault, or to participate in an offensive movement." With the arrangement proposed intelligent tactical use of the machine guns to meet the exigencies of the combat as they arise will be impossible.

The immobilization of the artillery is an even more serious mistake than that of the machine guns. The proper tactical use of field artillery requires that it be assembled in large masses (not less than a battalion and usually not less than a regiment) under the direct control of its proper commanders. If the artillery is immobilized by being distributed by battery on the line of the infantry defenses the control and direction of the artillery fire by the artillery commanders will be utterly out of the question. It will be impossible to concentrate the fire of the artillery on decisive points, and it can exercise no influence on the course of the action. The woods in which the battery shown on Plate I is located will probably greatly interfere with shrapnel fire, causing premature explosions, menacing the safety of the troops and betraying the position of the guns. It is evidently not contemplated to take any advantage of the relatively great range of these guns; the author, by so placing them, has turned his artillery into machine guns. Incidentally, they will also draw fire on the infantry near them, which is one of the reasons why the placing of infantry and field artillery in the same works is not ordinarily considered good practice. The remarks concerning the 3-inch guns apply with even greater force to the battery of 4.7-inch guns shown on Plate I. The range and power of these guns is so considerable, and the advantages resulting from their concentration under one control where their fire can be properly directed on decisive points is so great, that is it inconceivable that these advantages should be deliberately sacrificed by distributing these high power guns by battery within 400 yards of the infantry defenses, where they would exercise little more influence than machine guns. It is stated that the attacker is provided with nothing heavier than 3-inch guns. The possession of this heavy artillery should therefore give the defense an enormous advantage, if their fire were properly controlled, directed, and concentrated. It might be possible to prevent the attacker's artillery from coming into action within its effective range at all, or to crush

it it if it did so. The attacker's preliminary dispositions for the assault could also be greatly embarrassed by long range fire which his weaker ordnance would be utterly unable to check. These advantages will, however, be greatly diminished, if not entirely sacrificed, by a useless dispersion of these high power guns practically on the line of the infantry defenses, where also they are greatly and unnecessarily exposed.

* * * * * * *

The obsession of closed or half-closed works, and continuous parapets has caused the author to sacrifice frontal fire, to fail to properly cover his foreground, to immobilize his troops, machine guns and artillery, to expose a deep and vulnerable target to artillery fire, while arranging at the same time to draw this fire upon himself. These mistakes, for as such we regard them, are illustrated in all his dispositions, but especially in the two two-company works on the center and right of that portion of the line shown on Plate I.

LOCATION AND LENGTH OF FIRING TRENCHES.

The problem which the author has set himself is not to dispose his troops and prepare his defenses in such a manner as to effectively defend a given front with the minimum number of men, which is the essence of the defense, but to see how many men he can crowd into the firing trenches on a certain hill.

After laying down a trench which he believes will effectively cover the foreground, the author is confronted with the problem of immobilizing his superfluous troops (?). After crowding them as much as possible he still has men on hand, and being determined to have no regimental reserves but to place the entire regiment in trenches, he discovers two methods of accomplishing this undesirable end: (a) To lengthen his trench by placing it below the elevation he deems proper, thereby reducing the effectiveness of his fire, or (b) To extend the flanks of the trench to the rear, which he knows to be wrong. The troops in these extended flank trenches add little to the effective frontal fire, with which the enemy's advance must be met, but they furnish an excellent target for the hostile artillery. We propose a third solution, (c) Put as few men in the trenches as required to adequately defend the assigned front, and utilize the remainder where they are most needed—that is, as mobile local and general reserves under the higher commanders. Herein lies the essence of our criticism of this paper. We assume, in the absence of exact information on the point, that an active defense is contemplated, for such would be the usual case. What then, is the object of fortifications in the active defense of a position? To increase the effect of the defender's fire while diminishing the effect of the attacker's fire, in order to economize men on the firing line and in support, to the end that the defender may hold in hand large reserves for the decisive stages of the action and

for counter-attack. An active defense then, is characterized by the minimum firing line and support and the maximum mobile reserve. The author uses his fortifications for the very reverse of the purpose we have stated—to occlude or uselessly bury as many as possible of his troops.

The author has some difficulty with a dead space of about half an acre just in front of his work. If the defense were properly organized the attacker would probably never reach this point. Dead spaces are covered by a sub-division and distribution of force. The author either overlooks or chooses to ignore the fact that this so-called "dead space" is covered by the fire of the works just to the right, and at point-blank range. The problem was so easy that it solved itself, or rather there really was no dead space at all, only a bugaboo. We are glad that the author resisted the temptation to distort his fort in order to guard against this imaginary danger.

The author several times refers to the flanks of his work and is concerned with measures for their defense. It is evident from this that he does not regard the position as part of a general line of defense, since an interior portion of such a line can hardly be said to have flanks. He is contemplating an all-around defense, such as might be made by a small party of frontiersmen overtaken and surrounded by Indians. In a modern battle, however, it is out of the question to make every rifle trench capable of all-around defense and it would be an utter waste of time to attempt such a thing. Each section of the line supports and is supported by the sections adjacent to it. The works are so disposed that the front of the position is covered by frontal and oblique fire. So far from attempting to provide all-around defense it may be that some of the works can not even cover their own immediate fronts. In such cases provisions are made to protect such ground by the fire of adjacent works. This method of covering the *foreground* by a proper subdivision and distribution of troops and defensive works is one of the secrets of successful defense. The other great secret is the use of strong mobile reserves capable of being quickly concentrated on threatened points. In our judgment, the author has attempted to organize a defensive position with little or no regard for these most important principles. We have already referred to his method of using artillery whereby he does not allow the fullest possible scope to his own defensive weapons.

LOCATION OF SUPPORT TRENCHES.

The author seems to have a misconception of the use of supports. They are not generally employed to reinforce the firing line, but to replace casualties therein. It is not usual to hold out a portion of a company as a support—companies are sent complete into the firing line. Supports are not used for delivering counter-attacks—this is the function of the local and general reserves. In this case, the author has provided no regimental reserves. His dispositions of his support trenches are considerably influenced by the fear of

an attack *from his rear!* Dead spaces in the foreground may indeed be a cause for grave concern, but sheltered space *in rear* of one's line is generally spoken of as cover, and is supposed to be an advantage to the defense. Liable to be attacked in front, on either flank and even in rear, the defenders of this work are indeed in a trying situation.

Profiles of Firing Trenches. If the front slopes are actually 19 to 23 degrees as the author states, this is steeper than is desirable for fire trenches, except for long range fire.

Dressing Station. Our authorized medical organization does not contemplate a dressing station with operating table and other facilities for each two companies. Neither does it contemplate the performance of operations on the firing line. In case of a siege special dispositions would, of course, be made to meet the special situation. The aim of the sanitary organization is to evacuate all wounded to the rear as rapidly as possible. Hence at each successive station each man receives only such attention as is necessary to insure his safety while being moved back. On the firing line a man certainly would receive nothing more than first aid. Even at the regimental stations he would ordinarily receive only such attention in the shape of dressings as necessary before he was transported still further to the rear, although in a defensive position which was to be occupied for several days arrangements might be made to perform certain operations at the regimental stations. Two principal considerations determine the organization and methods of the sanitary service, viz: tactical and humanitarian. They unite in the demand that the wounded shall be evacuated as rapidly as possible to the large and fully equipped stations in rear. If each two companies were provided with a sanitary personnel and equipment sufficient to perform operations in the support trenches, as proposed by the author, the sanitary organization of the army would be a huge and unwieldy device which, instead of facilitating the tactical operations of the army, would be a drag on them. Such a distribution of the sanitary units is in line with the author's use of artillery to which we have referred.

Communications. The author is aware that closed redoubts, containing infantry and artillery are, except in very special cases, not nowadays in favor. He has therefore entitled his article, "A Two-Company Field Work." But what has he done? Has he not produced a deep closed work, delivering fire on all azimuths, and crowded with infantry and artillery? One face of this work he calls a "communicating trench," which, he says, "is designed for fire to the flank and *rear*." The change of name should not deceive him. This "communicating trench" is one of the faces of a closed redoubt.

Obstacles. The author says at the opening and again at the close of his paper that this study concerns a unit only, of a general defensive line. Yet he wants 21 miles of wire furnished for this 100-yard front of a long, general battle line. On this basis how

much would he use for his entire line, and where is all this wire to come from?

Time for Construction. The author assumes the output of a portable intrenching tool at 20 cubic feet per hour. We believe this to be excessive for a single hour. For sustained work it is altogether too high, even with reliefs, which are apparently not contemplated. It is proposed to work the men ten hours a day without relief. In an emergency well-disciplined soldiers will do this, without complaint, but their output would be greatly diminished. If the commander will give them no rest they will take it themselves.

The author has performed a valuable service to the profession in the attention he has given this problem, and in the presentation of this paper. The study of his work is profitable because he has exhibited in a single article virtually all the mistakes which are charged against what the French call the "polygonal" school of fortification. This is a study heretofore too much neglected by American engineers, and discussions on the subject must of necessity be beneficial to all. It will be apparent from these criticisms that the writers completely and frankly disagree with the author on most of the fundamental principles involved. The differences of opinion are too great to be harmonized. One party or the other is entirely wrong. Which?

Col. JOSEPH E. KUHN
Corps of Engineers

When employed in military operations, field works must conform to strategical and tactical requirements. The situation assumed in the present case is such that a large force, consisting of several divisions, finds it convenient for its purpose to take up a defensive position for the preparation of which two days are available. The particular section of this defensive line under consideration is the right half of a brigade sector, upon which one full regiment of twelve companies has been disposed in defensive works.

As shown in Plate I, the general line of works occupies the high ground bordering a valley, the floor of which, as marked by the stream, is 1,000 yards, more or less, distant from the line of works, with fairly open ground between. This constitutes a favorable defensive position, and the general location of the line leaves nothing to be desired.

The regimental front, measured between extreme works, is 2,000 yards, along which are placed eight separate works, calling for garrisons of from one-half to two companies each, located on all available spurs. While such a disposition will unquestionably give a most effective fire on all the foreground, it absorbs the entire twelve companies of the regiment, and leaves the regimental commander no reserve, the nearest troops for this purpose being the brigade reserve, which is some 2,000 yards distant from the right of the brigade sector.

While the tactical situation may be such as to require the regiment to defend a front of 2,000 yards, not an unduly excessive

front, the dispositions of the companies of the regiment imposed by the defensive works is open to criticism. To deprive the regimental commander of all reserves whatsoever practically eliminates him from the conduct of the fight in which his command may become engaged. Similarly, the absence of battalion reserves from the battalion groups leaves the conduct of the defense to the commanders of each of the works, with no means for actively supporting one another.

The tactical defects noted in the disposition of the troops can be readily overcome in several ways, either by reducing the size of the garrisons, thus setting free battalion and regimental reserves, or, the better way, reducing the number of works, or by a combination of the two.

As now located, the works would appear to be unnecessarily close together, the greatest interval being less than 400 yards, and the lay of the ground being such that a good defense can be over intervening ground with wider spacing. Thus, the two-company and one-company trenches between the extreme right flank work and the two-company work in the center might well be suppressed, as the ground is well covered otherwise. Similarly, on the left of the line, one of the two-company works may be omitted or replaced by one-company works. Likewise, the ravine for which two one-half company works are specially provided would appear to be sufficiently covered by the adjacent works.

The diminution in number of works, as above suggested, will probably result in some small dead spaces, but it is too much to expect that every square foot of ground in normal terrain can be reached by direct fire. Moreover, when the attempt to do this results in immobilizing the entire command and robbing the commander of the troops of every possibility of taking the initiative, it is not believed to be a safe procedure. The troops in the fire and support trenches are necessarily tied to their works; are, in fact, what their names imply, simply garrisons, and can not do more than defend their own works. With the disposition shown, the entire regiment is immobilized, and the possibility of an active defense, now recognized as indispensable, is out of the question.

Passing now to the technical features of the two-company work under consideration, they are in the main excellent and well planned to meet the requirements now deemed necessary. A work of this character would be very hard to damage seriously by artillery fire, either from light or heavy guns, owing to the dispersion of the trenches and accessories, and the insignificant targets offered. The details have been well considered, and while opinions may differ as to some features, the work as a whole is thought to be very satisfactory.

It is fairly open to argument whether the fire, communicating, and support trenches with a single line of entanglement could be completed by the garrison in one day of ten working hours. While the earthwork tasks are not unreasonable, there is a very large amount of covered construction requiring much timber and framing. Even assuming that there are sufficient cutting tools available to

supply this, its placing and covering will call for rehandling much of the earthwork. It is always well in the field to allow an ample margin of time for the execution of field works, which are executed under conditions widely different from the drill ground. It is also well to bear in mind that it does not pay to overwork men on the eve of a fight, and that due regard must be paid to the conservation of the physical and moral forces of the soldier.

While the task set the two companies assigned to the construction of the redoubt is probably somewhat excessive, there is no reason why the work should not be finished in the two days supposed available. By reducing the number of works, details from the regimental and battalion reserves, or even from the brigade reserve, can be called upon to assist and to lighten the labors of the garrison, which should be done if necessary.

The site selected for the 4.7 battery is a good one, as far as the battery itself is concerned. It is, however, deemed desirable to separate even more completely the artillery and infantry defense, by placing this battery farther to the rear, if suitable positions near roads are available. The long range of these guns enable them to be placed with great freedom without danger of sacrificing their efficiency.

Field problems of this character afford excellent training for engineer officers, and convey knowledge impossible to acquire from books. Rarely will it happen that actual topographical features permit of the complete realization of theoretical principles. It is in the adaptation to actual ground forms that one learns the limitations of fortifications, as well as their possibilities.

Maj. W. W. HARTS
Corps of Engineers; Member American Society Civil Engineers

The analysis of the problem of field entrenchment submitted by Captain Wilby seems carefully studied, but it is feared that the influence of the redoubt, or the old pattern of closed work, has been rather too strong in the design of the two-company field work proposed by him. Although there may sometimes be found positions requiring a work of this nature, it does not appear that the case presented is one where the type of work suggested is in any way desirable. This work is a part of a continuous front of a brigade in line, and is not an isolated entrenchment.

From the small scale map it is seen that the site, together with the salient hill, about 1,000 yards to the northwest, and with the hill about the same distance to the northeast present points where fortified pivots or supporting points for one or two companies each might be placed. The ground is found to be suitable upon examination, and these points can be definitely established as reasonable locations for supporting points. The portion of the line between must then conform to these guiding features.

The hill selected by Captain Wilby for the two-company work

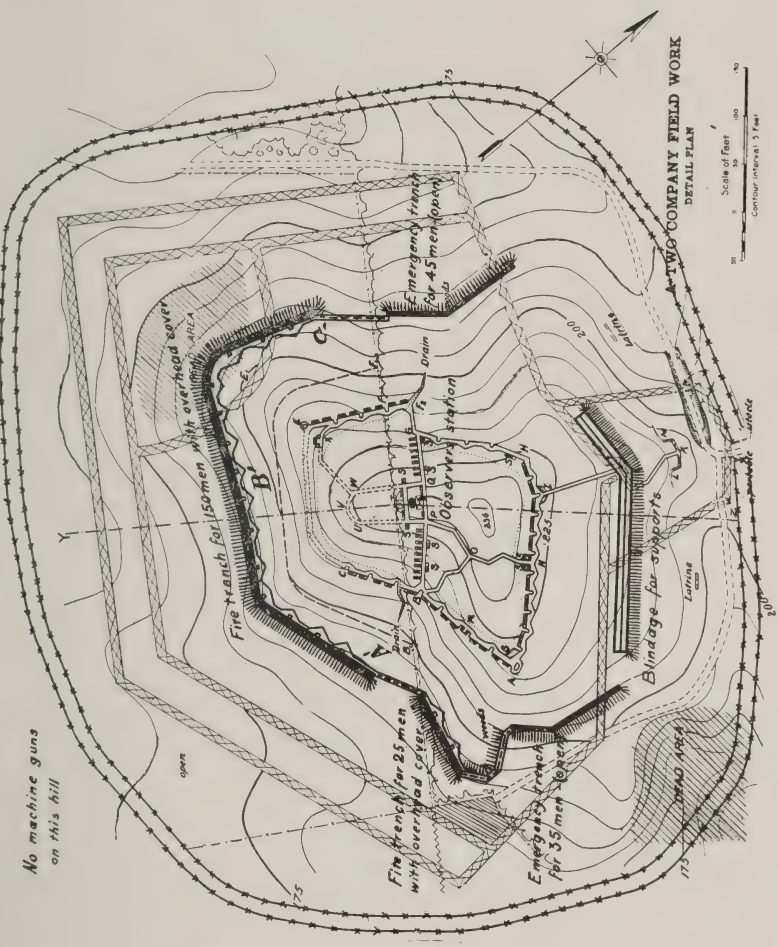


Fig. 10. Major Hart's plan in heavy black lines superimposed upon Captain Wilby's.

seems very cramped for so large a force as 256 men, unless the trenches are spread over more area than he indicates. It is usually desirable to have the front of any position accessible from other points of the line, and for this purpose connecting trenches are usually necessary along the front. They can often be placed where the section reserves or even general reserves may occupy them if found necessary to resist a specially severe assault.

This proposed work is to occupy the crown of a conspicuous hill in the edge of the wooods, the front slope of which is plowed and bright in color; it affords an easy target for distant artillery, visible from nearly every dominant point on the opposite ridge, which sooner or later will be open to the hostile artillery. The large number of men stationed on the crest of the hill would very likely make the casualties unduly large, notwithstanding the artificial cover proposed.

The shape of the work is longer in the direction of the enemy's fire than it is in a direction parallel to its own front, and most authorities now agree that such works should be as shallow in the direction of the enemy's fire as practicable. There is a lack of trench room for frontal fire, thus cramping the offensive power of the work. A shallow plan is better for the development of such fire, as well as for the better protection of the garrison. Both flanks, as proposed by Captain Wilby, would be exposed to a raking fire and even though concealed in the woods much damage might result.

It seems doubtful whether the location of the machine guns is the best available. The advance position at E is decidedly exposed and would probably be attacked and silenced at once by distant artillery the moment it opened fire. Better positions would be somewhere on the flanks or on adjacent noses. The artillery positions for the battery of 3-inch field guns are also too far to the front and too near to the infantry trenches. Artillery may always be expected to draw the artillery fire of the opponent, and if placed too near infantry positions may cause unnecessary losses. It should be withdrawn to the northwest and kept together.

Furthermore, the proposed field work seems to be needlessly complex, entailing more work than necessary. There are too many trenches and too many varieties of trenches. There are eighteen different profiles proposed. Simplicity is well believed to be a desirable thing in all engineering construction and particularly in field fortification.

A rough sketch is sent (p. 457) showing a suggested new arrangement of trenches, having in view the simplifying of the defense of this hill. It is thought that the woods should not be altered, except for an observation station on the crest of the hill, perhaps on a platform in a tree. The reserves should be held in rear of the hill, more than the depth of a shrapnel pattern from the front edge of the woods. A continuous trench along the adopted front seems necessary, part of which should be provided with overhead cover for perhaps one and a half companies in all. If one platoon only is on duty in the trenches before an attack is threat-

ened, the other platoon in support can prepare cover for itself somewhere behind the hill, near the main support. Barbed wire entanglement should be built wherever the work is easiest, without reference to straight lines, but it should not be placed closer to the firing trenches than about 50 yards, certainly not a less distance than a hand grenade can be thrown, and, of course, it will be always placed where it is under fire.

The trenches suggested herein are less compact, less conspicuous, and can probably be put in order for some fair measure of defense in very much less time and with much less work than those proposed by Captain Wilby. Would they not answer the purpose of protecting the troops better and of developing the maximum fire of the garrison more effectively?

Maj. Wm. D. Connor
Corps of Engineers, General Staff; Member American Society Civil Engineers

In connection with Captain Wilby's interesting discussion of the disposition of the half brigade, my differences of opinion are largely in details.

I submit herewith Fig. A (p. 461), my idea of the disposition of the regiment on the half brigade front shown. It seems as though an additional supporting point were necessary on the spur at the west end of the line, and I have shown a two-company supporting point there. I may be influenced in this decision by my knowledge of this particular section of the terrain, but I have endeavored to use only such information as is given on the map. The line I have selected is slightly lower down the slope than that chosen by Captain Wilby, and this location gives trenches more nearly straight and parallel with the general front than those of Captain Wilby's, and they yet are high enough to cover all the foreground.

I have not shown any support trenches, for, with the woods so close in rear, they did not seem necessary; but I show locations for battalion supports in rear of each battalion. Very properly, Captain Wilby shows trenches for *all* of the companies. At least twelve such sections of trench would be constructed for use when all the regiment was placed in the line; they would not all be occupied at first. The system of supports and regimental reserve would probably differ for the same ground with different regimental commanders. It is believed that there would be less mixing of different battalions and less marching involved in reinforcing the line by using the three battalion supports shown in Fig. A, but many officers prefer to hold out one battalion for a regimental reserve and assign the regimental front to the other two battalions, each of which will hold out one company as a support. However, this idea does not affect in any way the location or number of trenches along the front. Captain Wilby shows no obstacles in Plate I; they are easy to construct and may be of great value.

The machine guns are supposed to work in pairs, and have been

so assigned in Fig. A. Each location should be prepared with two two-gun positions, so that both guns can be used in either position if desired; they are placed in retired positions, so as to be less exposed to artillery attack. From these positions they are expected to be used against the flanks of advancing lines. Captain Wilby's positions seem too exposed and too open to the enemy's artillery, which will seek them out at the earliest possible moment.

I submit in addition sketch (Fig. B, p. 462) showing my ideas in regard to the supporting point. The trenches indicated give a greater frontal fire, and an equal fire to the flank and rear.

It is believed that the amount of work involved is no greater than in Captain Wilby's plan and that the work is of a simpler character.

The trench line AB, D^1E^1, and the front of BCD, will be occupied during the early part of the attack by about half of the garrison, the remainder being in the rear trench GH as a reserve.

If a close flank attack develops, the troops in AB and D^1E^1 will go into BCD and DF and BH and the troops in GH will extend into FD and HB. The last two-mentioned sections of trench will not be occupied until the flank attack becomes so close that the enemy's artillery must raise its fire to avoid hitting its own men.

A study of Figs. 5, 6, 16, 17, 19, and 20, Plate IV, leads to the belief that advantage has not been taken of the fact that all the position north of the wire fence lies in the woods and that sufficient concealment comes from the trees and brush. As far as practical, everything within those woods should be built above the ground level to avoid the troubles and discomfort arising from water in the trenches, and the small amount of room in excavated trenches. This can be accomplished by taking the earth for the trenches from in front of them.

The work required for the bombproofs GQ and BP in Plate II of Captain Wilby's plan might very readily have been combined with that necessary to construct the trenches and cover shown from A to H of same plate. The troops therein would still be sufficiently close to the firing line, and the shelter for the reserves would be combined with the trenches for firing to the rear. In Fig. B, GH is not only a shelter for the reserve, but is also built to afford a trench firing to the rear.

The trench BCD, Fig. B, will have a cross section the same as shown by Captain Wilby in Fig. 1 (Pl. IV). As far as practicable, the trenches AB, BH, and FD are to be constructed on top of the ground from a ditch in front of the parapet. There is no need of concealment, as they lie in the woods and the parapets can be as high as desired. This kind of parapet avoids wet trenches during and after rainstorms, and gives the increased room mentioned above.

The cross section of the trench GH, Fig. B, will be similar to that shown in Fig. 7 (Plate IV) if time permits. This cross section permits of greater comfort than can be provided by cross section shown in Fig. 6 (Plate IV), which would be constructed first if time was limited and could later be enlarged to the cross section shown

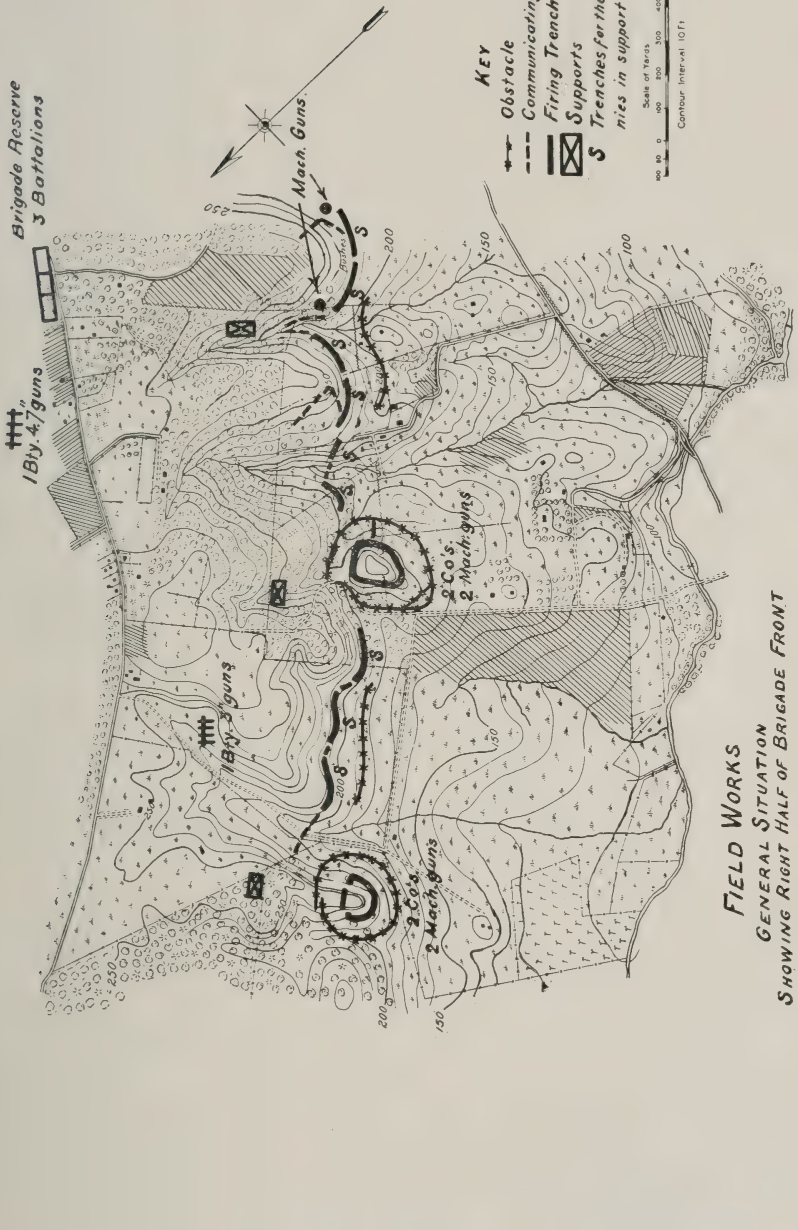
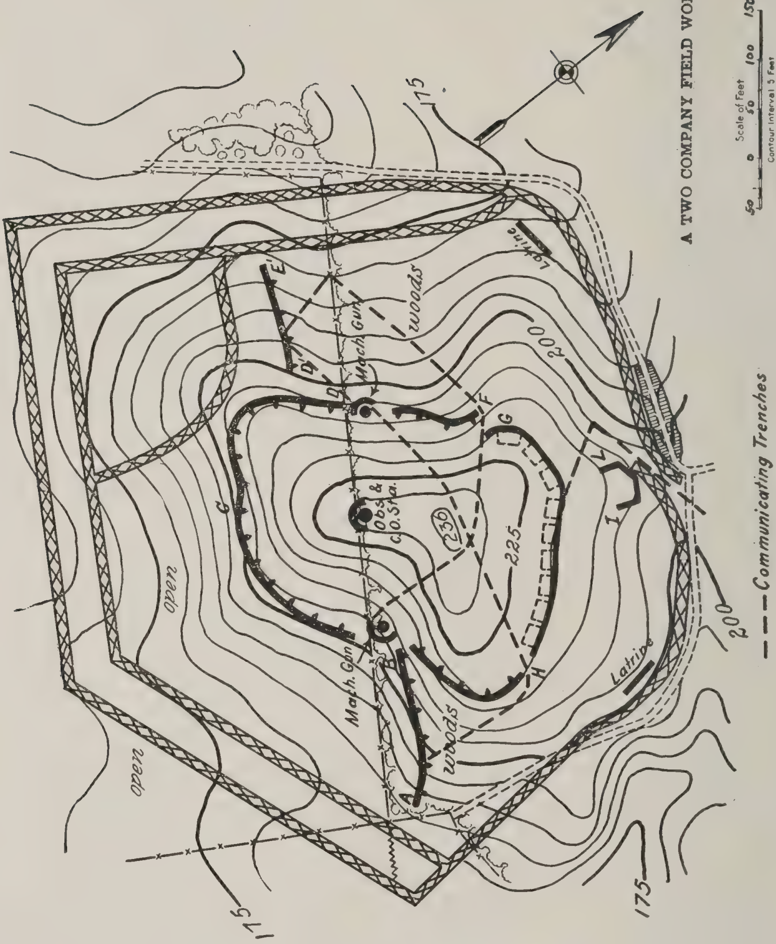

Brigade Reserve
3 Battalions
1 Bty. 4.7" guns
Mach. Guns.
Bushes
2 Co's.
2 Mach. guns
1 Bty. 3" guns
2 Co's.
2 Mach. guns
KEY
Obstacle
Communicating Trench
Firing Trench
Supports
S Trenches for the companies in support
Scale of Yards
Contour Interval 10 Ft.
FIELD WORKS
GENERAL SITUATION
SHOWING RIGHT HALF OF BRIGADE FRONT

Fig. A.



A TWO COMPANY FIELD WORK

Fig. B.

in Fig. 7 (Pl. IV) when time was available. Shrapnel will surely burst on entering the woods and shells probably will, so that protection against splinters and shrapnel is all that is necessary. Protection against howitzer fire is impractical and unnecessary in field fortification.

It is not believed that overhead cover is desirable along the front BCD, Fig. B. It would be plainly visible and none of the advantages of loop-holes or head-cover could compensate for thus advertising the location of the work. Absolute concealment of the works at this point is necessary, inasmuch as the hill is exceedingly conspicuous and draws attention of itself without any visible works. The only head-cover believed desirable would be at IL, Fig. B, a small trench used to cover the dead spaces in the adjacent ravines. In this trench head-cover seems necessary and allowable.

The 3-inch guns shown in Plate I appear to be too far to the front for an original position. I should prefer to have them on the open ground back near the road, as shown in Fig. A; from there they would fire by indirect laying along the front some distance to the right and left, while the immediate front of this brigade would be similarly protected by artillery fire from some other part of the line. The position shown by Captain Wilby would be better suited for a final position, in case of a stubbornly resisted attack where the artillery was brought up to the infantry line for direct firing. But, even in that case, it is an open question whether they would not lose more by moving at the critical time than they would gain from the new firing position.

The 4.7 guns have such a long range that they can well be located farther to the rear, and some place north of the road can be found for them. Such a site would be outside the limits of the map and so can not be discussed.

Lieut. Col. THOS. B. DUGAN *and* Maj. DE ROSEY C. CABELL
United States Cavalry

This paper has been considered under the following heads:

a. Should a supporting point be located at the point selected?
b. Character of work.
c. General plan.
d. Location and length of firing trenches.
e. Location of support trenches.
f. Profiles of firing trenches.
g. Profiles of support trenches.
h. Traverses.
i. Observation station.
k. Dressing station.
l. Connecting trenches.
m. Machine gun positions.
n. Drainage.
o. Latrines.
p. Obstacles.
q. Clearings.
r. Conclusions.

a. Should a supporting point be located at the place selected?

Assuming that there should be at least one supporting point on each regimental front, the location chosen for this supporting point seems the best. It is a salient, the only salient projecting beyond other lesser ones on the line and it therefore best supports with its fire the locations of the other trenches which must be occupied by the regiment.

b. Character of work.

The observations in the paper under consideration are approved; the work should be constructed to resist field artillery only—later being strengthened.

c. General plan.

It does not seem necessary nor desirable to make this a closed work. A firing trench somewhat lower down on the slope would give more fire on the enemy. Add to this a shelter for the supports behind the crest and a short connecting trench, and no others are considered necessary.

No rear fire would be necessary unless the enemy breaks through the lines, and if he succeeds in doing this in force the field of fire that can be obtained here is so limited and the ground that would be occupied by the enemy is so commanding that the place could not be held. It is therefore considered better to allow for as much fire to the front as possible, with a view to preventing a breaking through of the enemy in force. Should a small party break through, the supporting trench alone with fire to the rear will be sufficient to hold the place. This solution is simpler than that proposed and is considered better.

d. Location and length of firing trenches.

It is considered, as indicated above, that the firing trench proposed is too short, and as it can not be made longer except by lowering its position on the slope, that it should be lower down. The location recommended is shown on Plate II by the red line (same as main firing trench from A′B′C′ on illustration accompanying Major Hart's article [see Fig. 10, page 457]—Ed.). The length of this trench is, front, 85 yards; left flank, 45 yards; right flank, 50 yards; total, 180 yards, whereas the length of the firing trenches shown in the paper submitted is but 159 yards. It is thought that the lower trench has sufficient command and the dead space near E is eliminated.

e. Location of support trenches.

The location of the support trench is approved. If the work is not closed, this trench should be about 80 yards long.

f. Profiles of firing trenches.

The different profiles adopted in the paper are considered un-

necessary. One profile only, that of a simple standing trench is recommended. The men who will construct these trenches are probably not trained engineer soldiers. An overhead cover should be made if practicable later.

g. Profiles of support trenches.

The support trench shown in Fig. 6, Plate IV, is approved.

h. Traverses.

The question of traverses has been duly considered and well solved. However, should time permit some of the woods on the flanks and in the rear should be removed to improve the field of fire in those directions. This is almost imperative to the rear. As the woods form a curtain against which the burst of the projectiles of the enemy is readily observed, they should be allowed to remain standing only in event that they are regarded to be of necessary service for concealment.

There seems to be no good reason for making the passageways around the ends of the traverses so broad, as the distances within the work are so short that a wounded man need not be put on a litter to remove him to the first-aid station in the work.

i. Observation stations.

It is thought that sufficient provision is made for the commanding officer's supervision in the location of the three observing points. With the sheltered station at T temptation is offered the commanding officer to take up this safe position in which he will not come in personal contact with his men, a condition that should lessen the time that a determined resistance in the supporting point will be made.

k. Dressing station.

No dressing station should be established so far to the front. In consequence, a place for an operating table in the work will not be needed. A first-aid station might be established for the work at the place designated for the dressing station, but even this establishment, so far to the front, is open to objection. It would be of very limited usefulness.

l. Connecting trenches.

The provisions for such trenches seem to be adequate and their profile satisfactory.

m. Machine gun positions.

The location of the machine guns at E is regarded as excellent. The position at A, however, is not regarded as good, on account of the limitation of the field of fire that results from the standing woods. It is thought that a position near C would be a better first

position for the gun at A. The usefulness of the arm is much curtailed by the disposition made of it.

n. Drainage.

The means for drainage provided are necessary and adequate.

o. Latrines.

These are sufficient and well placed.

p. Obstacles.

The question of obstacles is well considered and solved.

q. Clearings.

It is thought that if time permits, all woods except such as are necessary to mask features of the work should be removed. This is particularly so to the rear of the work, where the sharp rise of the ground gives command of it within 500 yards.

r. Conclusions.

The work under consideration is regarded as being too elaborate. In it there are profiles of nineteen different forms, many of which would require in construction the superintendence of officers or enlisted men with greater technical ability than that possessed by the officers and enlisted men to whom this would in the ordinary case be entrusted.

Maj. M. L. Walker

Corps of Engineers; Member American Society Civil Engineers

Captain Wilby's paper on a "Two-Company Work," treating, as it does, of the defensive organization of an actual sector of terrain, as opposed to a purely theoretical demonstration, is worthy of much commendation, since it indicates the route over which all must travel to obtain real knowledge of the subject of field fortifications. This subject has been largely neglected by American military engineers, and articles leading to its discussion must be beneficial to the service.

It is generally accepted that field fortifications must conform to the plans of the supreme commander, and must not be so designed as to unduly influence his action. It is to be regretted that in Captain Wilby's paper the military situation is not given in sufficient detail to permit a discussion of the dispositions from this viewpoint.

In the time at my disposal since the receipt of the advance copy, it has not been possible to make a thorough critical study of the paper, but certain features have at once struck me as not being in agreement with the present generally accepted principles of tactics, to which all fortifications, if they are to be of maximum efficiency, must conform.

There is no difference of opinion among military men as to the

soundness of the dictum that "the offensive alone secures decisive results." The organization under discussion shows practically a continuous line of infantry trenches—the regiment having placed all battalions in the line, and each battalion having assigned all companies to the fire trenches, the companies individually holding out supports. The regimental commander having no reserve in hand, is eliminated from any control of the action, and the battalion commanders, having no supports subject to their orders, are in the same plight. Consequently, the fight becomes a company commander's affair and no opportunity for offensive action, however brilliant the possibilities, can be taken advantage of, owing to the lack of general direction.

From the map, there is no reason apparent why two battalion supporting points should not satisfactorily cover the sector under consideration. Had this been done, the majors holding supports under their orders and the regimental commander having the remaining battalion under his control, the tactical requirements would have been met, and means would have been provided for passing to the offensive should opportunity offer.

In the sector under discussion, nine machine guns are emplaced. It is generally believed that six machine guns is the maximum suited to a regiment, and our Service deems that even less than this number should be so assigned. The troops at disposal in the problem are supposed to be in the field with their ordinary equipment, and it is thought that an element of unreality is introduced into the solution when their probable armament is exceeded.

The use of an entire battery of field artillery as a "stabbing battery," for sweeping the regimental front, is deemed extremely objectionable. The number of guns in a command will scarcely exceed 4 per 1,000 men, and any such use of them as set forth in the paper under discussion will practically fritter away the artillery without attention to its legitimate function of distant defense. This use of artillery is exceptional—as, for instance, to cover a line of approach for the enemy which it is otherwise difficult to care for. The term, "stabbing battery," is rather catchy and is apt to make one forget that guns so used are out of the general action, and if the special necessity for which they are emplaced does not arise they play no part.

The location of the battery of 4.7-inch guns within 400 yards of the firing line is open to objection. These guns, while capable of being moved, are scarcely mobile artillery, and advantage should be taken of their long range to place them where they will be effective and still comparatively safe. The paper contemplates the possibility, nay, even the probability, of the line being pierced, and the occupation of the high ground in rear by hostile troops. In this event, what becomes of the 4.7-inch guns?

To pass now to the "Two-Company Field Work," we find several features to which exception is taken:

a. In the state of excessive anxiety about the flanks, frontal fire

has been sacrificed to a most objectionable extent, only nine squads out of thirty-two delivering fire where it is really most desired. The protection of flanks should properly be secured by the echeloning of troops to the rear, desired volume of frontal fire being thereby retained.

b. The extension of the flank trenches, together with the closure of the work and the practical emplacement of field artillery therein, presents exactly the kind of target which will please the hostile artillery, be disastrous for our side, and which violates the generally accepted ideas that guns should not be so placed as to draw fire on infantry and that the depth of target presented should always be a minimum.

c. Defense of intervals and rear is arranged for as if this were an isolated work, no reliance apparently being placed upon the cooperation of adjacent works, nor upon the action of reserves. This last, of course, is natural since no local reserves were provided.

The criticism has often, and sometimes justifiably, been made of the defensive organizations of engineers, that tactical requirements have been ignored; that the view of the engineer has not been sufficiently comprehensive. Fortifications, especially in the field, are not a thing apart, but are merely one of the elements of the military art. Their use is at times indispensable, but they must always bear their proper relation to the other constituents which go to make up a satisfactory military whole. It appears to the undersigned that the paper under discussion is particularly valuable in bringing these facts into prominence, and that a critical study thereof will be of great benefit to our branch.

Maj. Charles Gerhardt
United States Infantry

In consideration of the two-company Wilby field work, the following points offer:

1. The details are too complicated for easy construction by troops. The type should be simple and with few refinements.
2. The trace gives too little frontal fire in proportion to interior area covered and amount of work involved.
3. Traverses take up too much room. Not more than one for every two squads seem necessary and probably less would serve in the frontal element.
4. The geometric straight line for wire obstacles is not favored. A sweeping curve seems better.
5. A lower position on the hill is thought better for the trace, first fixing the frontal fire trench favorably, then extending by a communicating trench (improved for firing) on the refused flanks, with a support trench straight across the rear for cover and for fire to rear. Wire obstacle all around.
6. Placing a machine gun at E to cover a dead area with a de-

flector in front seems not so good as covering the dead area by fire from a neighboring trench.

SUMMARY.

A work constructed by troops is almost certain to be divested of complicated details that are usually present in a theoretical study, hence it is reasonable to recognize the fact in working out the study and strive for simplicity.

Maj. LYTLE BROWN
Corps of Engineers

1. There are very few criticisms that can be made of the two-company work as it has been laid down. It would be the most visible work on the front and would attract much field artillery fire, but the assumption as to the caliber of the opposing guns robs them of most of their power to harm the work, as it would stand in accordance with the plans shown. There are some minor points about the work that may be it would be well to consider. The latrines, if intended for the work, should not be outside the obstacle. The dressing station is so contracted that the seats should be removed therefrom; facing the operating table, as they do, at a distance of 2 feet would make them not a good place for wounded men in waiting. The wire entanglement on the near side of the dead space off the S.W. salient would be of more service if used to thicken the obstacle on the far side of the dead space. Great care should be taken that there is sufficient head room under the overhead cover of the firing trench, particularly where the immediate foreground falls away rapidly; it should not be much under 6 feet. While the two machine guns are well placed at diagonally opposite corners of the work, it would be well to have other and differently located emplacements prepared, into which they could be placed, the one in the woods might not come into action when much needed or when the other had been disabled. The kind of soil into which the excavations for this work are made is not mentioned; there are many steep slopes and, in some instances, the crests of these slopes bear large weights due to the overhead cover; in any but extraordinary soils the engineer officer would, and should at the outset, be confronted with the question of revetment and the labor and material involved therein. The question of time and labor, taken into consideration together with the situation to be met, is of large influence in determining what shall be done in any case. Before the decision to build any kind of work is formed, the time must be estimated for completion and compared with the time that the general or particular situation promises as available. Rarely will we be able to do just what our sense of the most desirable thing to do would suggest.

2. In this magazine I have seen other discussions of "closed

works,'' works for ''all around'' defense, sometimes called redoubts. The discussion, as remembered, seemed to have taken the turn that such things are out-of-date, not modern, and not good. But I do not remember the reasons cited in support. May be they had as their basis the accuracy and power of modern artillery. Whatever the reasons were, they were not so put as to carry any conviction with them. Modern artillery or any other modern contrivance is only feared when the enemy has more of it and of better stuff than we have. Match them up and other things which were once good are likely to be good again, when used with the proper judgment.

3. Closed works of the kind shown would rarely be found in the defensive positions of a mobile army; these positions had been forecasted long in advance of their occupation. They can not in America be so forecasted. An active force in the field is something like a good man in the ring, his footwork counts for much and he can hit from any angle; there is no posture his adversary can adopt and still feel safe; a studied and laborious defense is not worth its effort. So with the building of redoubts: the labor, when taken into consideration with the uncertainty of the actions of the enemy, is not worth its doing; such will, I think, be the view of the directing authorities in most cases. There must be emphatic reasons, apparent to the ordinary common sense, before an engineer recommends an excess of work at any point. The commanding general might say to his engineer in this case (Plate I) ''Why, sir, do you want to build this work at this place on the sector when there are four other places that it may just as well be built?'' or ''Why, if this work is good here, do you not build others of the same kind at the other four places?'' The answer must be sound, free from fancy, and the reasons absolutely conclusive to any sensible man. The reasons for building closed works are the same now as they used to be (and can be found in the books) when the situation demands the works should be built, and built to correspond with the power of the arms that are to be used against them. The fine points and close discriminations are worth in the construction of field works about what they are worth everywhere else, very little.

4. No map is as good as the ground, therefore I would hesitate to find fault with the arrangement on the sector of the works (Plate I). At a glance I would say that for the case of ordinary field defense there is too much seeking for flank effect at the sacrifice of front effect. Where practicable, I would prefer the zone of densest rifle fire to be from 500 to 800 yards from the general front, with the view of forcing the attack into his first lodgment as far from the line as the power of our rifle will permit; its point-blank range is about 500 yards, its fire from trenches, well covered, will be very effective up to 800 yards. The close flank defense should be attained by the machine guns.

Capt. C. O. SHERRILL
Corps of Engineers

This study is a considerable advance in method of treatment of the subject over the article published on pages 275-281 of Volume IV of the PROFESSIONAL MEMOIRS, especially in that the work on which this article is based takes into consideration the tactical situation and the work is planned to fit this as well as the special topography of the site.

There can be no more valuable training in field engineering for engineer officers than the practice such as was had by the officers of the First Battalion of Engineers in development of this work. A similar method was used in my classes at the School of the Line from 1908 to 1911, consisting in stating for each student officer the general and special situations; each would then independently solve the problem as a whole and choose his line of defense, working out the details on a combined sketch previously made by the members of the class. These student officers being in the cavalry, infantry, or artillery, were not required to take up the construction features in the minute details desirable for engineer officers. Engineer officers, however, should go thoroughly into the tactical solution of the problem as a preliminary to the more detailed consideration of a selected work chosen from the best tactical solution.

In the author's paper he gives something of the tactical situation, but it appears to me to be advisable to extend the scope of this feature, so as to have clearly understood the plans of the commander, the objects to be secured by his troops, the size and composition of the enemy's forces—in other words, a complete statement of the general and special situation. It would also add to the interest if a small scale map of a portion of the theater of operation should be included to indicate reasons for occupation of the chosen line and its defensive possibilities.

The position selected for the two-company work seems to be only fairly well adapted to all-round defense, due to the fact that an enemy occupying the commanding spur about 300 yards northwest would render it untenable. Another objection to the site is the shape, which makes necessary a considerable depth as compared with its front. As far as can be determined from the map, which, of course, does not show all the controlling minor features of the ground, it appears that a better arrangement for a smaller force could be made, say, for one company.

The location of the battery so close to the flanks and rear of the work appears to be open to criticism, as the artillery fire drawn by this battery would be dangerously near the infantry trenches, with resulting danger and demoralization to the riflemen. Also, unless the woods are very sparse, the fire of the guns would be interfered with by the trees. The entire artillery force massed at the site of 4.7-inch battery could apparently cover the entire front to good advantage, and be more effectively handled and be in a safer position in case of flank or rear movement of the enemy.

The types of trenches used are good, but in some cases a large

amount of rehandling of earth is necessary, as, for instance, in the trench from C to E, due to the type used.

It is to be hoped that similar studies to the one under discussion will be undertaken frequently for the general benefit of all officers.

Capt. W. G. CAPLES
Corps of Engineers

GENERAL SHAPE OF WORKS.

The front is generally southwest. An attack must come from this direction or expose its flank. The works follow the contour of the ground so closely that in some cases not more than a third of the trenches fire to the southwest, while the long returns are open to enfilade from that direction. It would seem better not to follow the contours quite so closely and to give more fire to the front, while offering less chance for enfilade by extending the works parallel to the front and suppressing the long returns.

LENGTH OF FIRING CREST.

The firing crest of an inclosed work should provide a yard of front for every man in the garrison. Less space hampers men in the use of their weapons, prevents the use of headcover for all men firing, and prevents obtaining proper volume of fire. One man every 4 or 5 feet will stop the most determined attack, while even greater intervals will suffice for an ordinary assault. It is the infantry fire that determines the final result; as many rifles as possible must be brought on the line to stop an attack, but the rifles are useless if the crest does not permit of their free use and the protection of their users. Short lines invite envelopment, while long ones make it exceedingly difficult. Counter-attacks by a force in an inclosed work are scarcely possible, for the obstacles would aid the enemy instead of the garrison, which, if defeated, could scarcely regain the work in condition to continue its defense. The garrison of the works deliver fire against the attack; the local reserves make counter-attacks, by passing through the intervals between works, and feed into depleted garrisons of works; the division reserves cover threatened points and make counter-attacks; the general reserve guards against enveloping attacks and trys to make them against the enemy. Such are broadly the functions of the different parts of the line. Not only must the garrison of the works be provided with 1 yard of firing crest for every man but some trenches must be added in the intervals to accommodate reserves brought up to aid threatened points. The deployment shown on the plans appears to be only about half enough; in fact, the work can probably be held by one company quite as strongly as by two.

LOCATION OF BOMBPROOFS.

The increasing accuracy of artillery fire has enabled the attack to keep down the rifle fire of the defense up to very short ranges. In

fact, some writers propose that the artillery continue firing until the attacking infantry is within 50 yards of the works—rather an extreme proposal, but one that shows the tendency in modern tactics. To meet such conditions the defenders have but little time to leave cover and man the works. The present tendency is to bring the bombproofs close to the firing line, some even placing them under the parapet. It would seem better to move the bombproofs up closer to the firing trenches even at the risk of an occasional hit. War can not be made entirely safe.

TYPE OF FIRING TRENCH.

If shrapnel can enter a trench, flesh and blood must leave. The increasing angle and accuracy of artillery fire has made the chances of getting shrapnel into a trench greater and greater. Following on the heels of the Deport gun, firing at 60 degrees elevation, comes the Polte cartridge, which permits of a variable charge with fixed ammunition. It seems certain that in the next war trenches will be searched in a manner of which the Russo-Japanese War was merely a suggestion. Headcover, shown then to be essential, may not have to be supplemented by over-head cover. It will not do to keep the rifles buried until the artillery ceases firing; they must be on the line every minute opposing the enemy's infantry advance and it is the duty of the engineer to devise trenches that can keep them there. A promising type is the one used by Maj. Lytle Brown at Pine Camp some years since. Squad or half-squad trenches are dug with natural traverses, which are carried up 1 or 2 feet. Plank or poles with intermediate supports are laid from traverse to traverse and sand-bag loop-holes are made between traverses. The parapet is carried up over the traverses and forming over-head cover. A deep passage is dug just in rear. If the parapet is not steeper than 1:10 with the natural surface of the ground and the loop-holes are judiciously screened, this trench is quite well concealed. The advantages of this trench are that it combines at once head-cover, over-head cover, splinter-proof shelter, traverses, and lateral communication in the minimum of time and with the minimum depth from front to rear. Only a shell or a shrapnel case can penetrate the cover, while the chances of a bullet getting through a loophole are decidedly less than of its hitting a head above a crest even when the loopholes are quite visible. Men in a trench of this type can deliver effective fire when men in an open trench would be driven into the bombproofs. Loopholes do render a trench more visible, but this disadvantage is largely offset by the superior protection afforded; while one or two rows of corn, a mask of weeds, or any other device that will blend in with the rest of the landscape makes them very difficult to see, it is not quite invisible. It would seem better to provide head-cover at least and probably over-head cover for the whole firing trench. The houses, woods, and hedges between the work and the creek must be removed anyhow and will furnish the material needed.

NUMBER OF TYPES.

The infantry have to dig the trenches, a duty that occupies only one phase of their training. The fewer and simpler the types the better the work will be done. One type of firing trench, another of bomb-proof, another of connecting trench are about all that should be required. All accessories such as latrines, dressing stations, kitchens, etc., should be simple rectangular excavations, roofed over if need be, but otherwise unadorned. A multiplicity of types is liable to cause confusion.

OBSTACLES.

The double obstacle is justified by the fact that, if defended, a three-row entanglement is almost impregnable, but the defenders are prone to leave when they see the attack cutting through the last barrier between them and the bayonet and fail to properly use the obstacle. A second obstacle—even a simple fence—helps by encouraging the defenders to make full use of the main obstacle. Where wire is limited and the men green, the apron fence (Figs. 102 and 103, p. 386, E. F. M.) furnishes a convenient and effective obstacle. Captain Wilby's remarks are quite apropos. Where is the wire coming from, since none is carried with the troops and a good many miles of wire fence are required for an entanglement? A parapet concealing the obstacle is needed to protect it from hostile artillery and to add the element of a surprise for the attack.

RANDOM OBSERVATIONS.

The field of fire to the creek should be cleared thoroughly, leaving only such natural screens as hinder the enemy's observation without giving protection from the defender's fire. Latrines should be close to the trenches because men will not walk far to get to one under fire. Dead spaces near the works should be covered with mines laid by the engineers. A trench for reserves just in rear of the hill would make a less elaborate gorge defense possible. Several parts of the line are well adapted to the use of dummy trenches.

Capt. A. B. Barber
Corps of Engineers

It is much to be desired that our infantry, assisted by our engineers, should be able to fortify a defensive position with works as good in design and detail as Captain Wilby's "Two-Company Field Work." This article shows clearly the large amount of special design necessary to adapt properly the ring trench idea to an average site. If detailed plans had not been worked out in advance, average troops of our army would be unable to produce in two days a result at all comparable with this field work. This fact is pointed out as showing the need of systematic instruction of our officers and troops, line as well as engineers, in design and construction of field fortifications.

The discussion of open versus loopholed trenches is based on the correct consideration for this case—that is, visibility, in which the chief factor is slope. In the general case, however, the tactical and technical considerations which determine whether or not loopholes and overhead cover should be used are so numerous that general rules, such as "17° to 18° of slope," must be used with great care.

Under "Location and Length of Firing Trenches" it seems to be assumed that two companies must be placed in the field work. To accommodate the two companies the author is forced into a difficult situation. First, he takes advantage of our obsolete system (which fails to provide automatically for replacement of losses) to discount the strength of companies; second, he introduces the incorrect tactics of holding out part of the *support* for *counter-attacks* (these should be made by troops outside the work and the entire support be considered available to replace losses in the firing trenches) and finally he resorts to the expedient of lengthening the flanks apparently without *previously* considering on what part of the foreground the available rifles should be brought to bear. The result, as the author later points out, is good, for it happens that more rifles are needed on the flanks.

The above remarks are made to invite attention to the difficulty of assigning troops properly to positions in a defensive line. The usual method is to throw them in, the larger the site the more troops assigned, a method undoubtedly necessary in hasty work but to be avoided when time allows study of the defensive organization and full employment of "the art of the engineer."

Much attention is given abroad to coordination and mutual support of the various elements of defense, and the method of design often adopted for that purpose is at least useful in turning attention in the right direction.

The general line having been selected and probable sites of works and trenches picked out, the foreground within the important zone of rifle fire (this zone varies with the character of the defense to be made and other considerations) is divided into sections by lines running to the front (not, however, usually perpendicular to the general front). The section lines roughly follow the limits of areas visible from the more important sites on the line. The relative strength of the hostile attack over the various sections is then estimated, due consideration being given to terrain, effect of artillery fire, etc. The available or necessary number of rifles (squads or platoons) is then apportioned to fire on the various sections in proportion to the needs of each and locations for the firing trenches are selected. Reasonable care is, of course, taken to avoid splitting units, and any surplus of men is carefully held in reserve. Tabulation and use of a graphical method for indicating cross fires facilitates the analysis and affords a ready check. In some cases all works out smoothly and sites are readily found in the right locations to meet every need, chiefly by fire directly to

the front. In other cases, it is extremely difficult to bring the necessary amount of fire to bear on some particular section and extensive cross fires have to be provided. Often also, as at the corner E of Captain Wilby's field work, additional strength must be provided by use of special obstacles or other special constructions, sometimes, even in this day, two tiers of fire.

This analysis of the field of fire is only an elaboration of what is usually in a small way done by *coup d'oeil*. In the writer's observation of field fortification studies reliable results are generally obtained more quickly by analysis. Checked up by analysis, as far as can be done on the map, Captain Wilby's disposition of troops appears to be excellent and leads one to question whether, though the reverse is inferred from his article, Captain Wilby did not apply deliberate analysis to the problem of disposing his troops.

Comments on a Two-Company Field Work and Preceding Discussions

Maj. AMOS A. FRIES
Corps of Engineers; Member American Society Civil Engineers

When a man sets himself up as a target he must expect to get hit, and the more prominent the target the oftener and the harder will he be hit. But target practice is always a good thing for a hunter, and one can not have target practice without a target. Now, if the target is such that practice on it approaches closely to the conditions to be encountered when shooting at game, the practice becomes of the utmost value to the hunter.

Captain Wilby in his article on a "Two-Company Field Work" has furnished exactly the target most sorely needed by the Corps of Engineers to-day. Once, when the writer as a cadet at West Point was making a bluff at reciting on the Roman legion and the Grecian phalanx, the instructor remarked, "It sometimes pays to indulge in glittering generalities, but you have gone too far; that will do."

A careful reading of Captain Wilby's article and the comments accompanying it shows what glittering generalities the Corps of Engineers is to-day indulging in concerning field fortifications.

If this discussion serves to crystallize the ideas of the Corps on what are the best types of works for field fortifications, the proper locations for them and the times when each should be used, it will have aided powerfully in preparing the Corps to assume in any future National struggle for existence the proud position it held during and at the close of the Civil War.

When one has arrived at the above conclusion he at once asks himself, What is the reason for such a variety of opinions on a single subject among men educated at the same schools and following the same line of work?

In the writer's opinion it means just one of two things, either

a lack of thought and study, or poor instruction. (Did someone suggest that it might mean both?) No matter, the result is the same in either case, and the remedy is study and free, unbiased discussion.

Every man can afford to change his opinion at any time. The thing he can not afford, is to be wrong.

Then let us drop all personal hobbies, and go at this with a determination to stay with it until it is worked out and worked out *RIGHT!*

Now, another question naturally arises. What must a man know to be a good military engineer?

He must be a student of war, and the conduct of war in all its phases. He must know not only how to plan, lay out, and build every work of fortification and their accessories, but he must know infantry tactics as well as the infantryman, artillery tactics as well as the artilleryman, and a good general knowledge of the proper functions of all other branches of the army—cavalry, signal corps, Quartermaster Department, Ordnance, and last but not least, aviation.

The need for the military engineer to make deep study of infantry and artillery tactics should be strongly emphasized. The very term has come to mean, among other things, one who plans, lays, and constructs positions in the field for the infantry and artillery, or at least one who takes the most prominent part in such operations.

Can he do this properly without knowing fully the capabilities and the limitations of those arms? He can not!

Now, having the range and windage for Captain Wilby's target, it is time to try for a bull's-eye.

It is a *sine que non* of good engineering to-day that the simplest construction which will do the work required of it is the best. If that is true of civil engineering during peace and plenty, how enormously more important it is in war when on the proper utilization of moments hangs the fate of nations.

Captain Wilby shows nineteen different cross-sectional profiles and six different special plans for one field work. These, including the wire entanglement, are to be completed in two days by two infantry companies.

The writer seriously questions the ability of two companies to do the work according to all those special designs in twice the time.

In actual practice not to exceed one-fourth of these special designs would be used, and all dimensions would be rough and approximate.

The answer will probably be that the author would have been criticised just as much had he failed to work out carefully all the details shown. By the pedant, yes; by the practical man, no.

As examples of various types of construction ideal for use under the varying conditions found at the site of the work the article is excellent. Moreover, they would not have been out of place had they

been appended to a simple solution of the problem, providing that sufficient emphasis were laid on the fact that they were types only and that it could not be expected that they would all be used unless a long lull occurred in field operations, when the work could be completed so as to become a permanent fortification in everything except durability of materials, as at Liao Yang or before Mukden in the Russo-Japanese War.

Captain Wilby assumes that two full days will be available for the work.

It is questionable if he has a right to make such an assumption and call the work a "Field Work."

From a fairly intimate knowledge of the conditions surrounding the making of the "position sketch," and the later working up of special details by Captain Wilby, the writer is of the opinion that ideas of "land defense" of seacoast fortifications and of cities and harbors was allowed to overshadow "field" conditions.

The two must not be confounded. The term "land defense" in this country now means deliberate defense.

Field fortification is not deliberate, except under special conditions which must never be counted upon.

In land defense careful surveys are made during peace, sites for "redoubts" and trenches are marked out on the ground, and on maps, all works including clearings, obstructions, etc., are worked up in detail and the complete plans filed with the district engineer for instant use whenever it is deemed probable that the place will be attacked.

It ought to be safe to assume that the engineer will have at the very least a week for construction purposes. If he has not, woe be unto us with our present forces. Anyhow, if less time than a week is available all operations can be classed as "field" operations and the need for simplicity in construction will be as great or greater than in any other such operations.

Every description of a field work should be preceded with an admonition that all firing trenches must be the simplest that are capable of giving progressive protection until at least a simple standing trench is reached and which can be later widened and deepened to afford passageways in rear, when if time is available they may be altered to give overhead cover and be loopholed and otherwise fitted to withstand a long, determined attack.

In field operations the question of whether or not a given trench is 10 yards too high or too low on a hill is of insignificant importance compared with the necessity for getting a relatively strong trench in the minimum time and with the minimum labor.

Conservation of resources in peace is important, in war it is vital.

Men must not be unduly fatigued before the final struggle that means victory or defeat.

And yet they must intrench, not only often but at every halt in the face of the enemy. Hence the imperative importance of simplicity in trench design.

Every officer and soldier in the army who is liable to get onto

the firing line is concerned in the cross section of the trenches to be built.

In addition, Engineer officers are particularly interested in the proper location of all trenches.

From the foregoing discussion it might seem that the proper type of trench was the most important thing in the Army. It is, but only from the point of view of its universal application.

A type trench which per yard takes only a little more work than another which will serve the purpose equally as well, will result in a campaign by a large army in an enormous waste of physical energy, perhaps the most valuable possession of an army.

On the other hand, the *proper location* of field works is *inherently* one of the most difficult problems any officer is ever called upon to solve.

The problem is here considered in its broadest sense. Given a certain hill to fortify and the problem is 95 per cent solved.

To properly locate field works requires a broad knowledge of tactics and strategy in general, and deep and accurate appreciation of the strategy of the campaign for which they are constructed. This involved a knowledge of the tactics of the commanding general, and the general tactics, strategy, processes of mind, and physical and mental characteristics of the officers and men comprising the enemy's army.

Added to the above must be a thorough knowledge of all the arms of one's own army and those of the enemy and of the skill and ability of each combatant in the use of the arms he possesses.

Truly, the need of knowledge by the military engineer is great, and if these remarks will help to make engineers feel that need and try to supply it they will have been worth while.

Skill in the location of field works can only be acquired by much practice, both on the ground and on maps, but particularly on the ground. How many Engineer officers have laid out at one time a mile of field works on the ground?

Discussion

Capt. F. B. WILBY
Corps of Engineers

In commenting on the above discussions, which have kindly been referred to me in advance by the editors, I wish first to express my appreciation of the interest shown by the numerous discussions submitted and then to attempt to clear up some misunderstandings which I believe my readers have been led into by lack of clearness in the original article.

ORGANIZATION OF THE BRIGADE FRONT.

The title of the paper under discussion is "A Two-Company Field Work," the design of which might properly be assigned to a captain or lieutenant of Engineers, or possibly to the infantry officers of the battalion occupying the section. *So far as these officers are*

concerned, the general situation is believed to be fully stated in the text, and, as given concisely by Colonel Kuhn in his discussion, "is such that a large force, consisting of several divisions, finds it convenient for its purpose to take up a defensive position for the preparation of which two days are available."

It is believed that this, with the general location of the line, the assignment of a sector for defense, and the knowledge of the direction of probable attack and location of supporting troops would be all the strategical and tactical information available for the officers concerned with the design of this *Two-Company Work.* The plans of the division commander, and especially of the army commander, as to any offensive returns or counter-attacks, can probably not be made at least two days in advance of the appearance of the enemy, and even if planned so far ahead it is not believed that every captain of Engineers or Infantry would be so informed.

There is a tendency to-day in the theoretical study of the art of war to place a major-general's information at the disposal of every captain and major of the line. Of course, that condition is greatly to be desired, but it is believed the studies would be made more practical by the introduction of a little more of "the fog of war" and by giving to each officer only such information as he would probably have under service conditions.

The problem before us then was the proper preparation for defense of a certain hill with *two companies* to furnish the firing line and supports, and *not* the dispositions of the *brigade*, or of the *artillery,* or should we be concerned with the offensive plans of the army commander. Perhaps with the selection of this hill for defense, the problem "was 95 per cent solved," but it did not seem so to the writer, and the other 5 per cent was the only part of the problem which was intended to be covered in the original article.

Enough of the original position was shown on Plate I to show the relation of our two-company work to the remainder of the line, but the brigade position was barely referred to in the text, and was shown *purposely incomplete,* omitting the *division into sectors,* the *field of fire from each trench,* all *obstacles,* most of the *artillery,* all *new roads,* location of *camps for the supports,* and *dummy trenches,* all of which appeared on the original solution and were omitted from Plate I as being unnecessary for the proper solution of the two-company problem, and liable to lead discussion away from the disposition of our two companies to the larger problem of the brigade.

However, for the benefit of those who are interested in the brigade problem, Plate I shows the front of a *regiment and a half.* All three regiments of the brigade are on the line, each regiment holding out one battalion as local reserves (these reserves being held at brigade headquarters until the attack develops). This leaves six battalions available for the firing line and supports, for the entire brigade front, each company furnishing its own supports, *all in accordance with paragraph 278, Field Service Regulations, 1910.*

COMPLEXITY OF THE DRAWINGS.

The problem which the writer attempted to solve was the proper *location* and *design* of the defenses necessary for a certain locality. If he had made a rough sketch of his most important locations (see Fig. C) and had copied three accepted types of trenches out of the standard works on fortification, he would apparently have met with more approval from some of his readers, in spite of the fact that *none of these standard profiles would have been suitable for the site on which it was to be built*, and, if built without modification

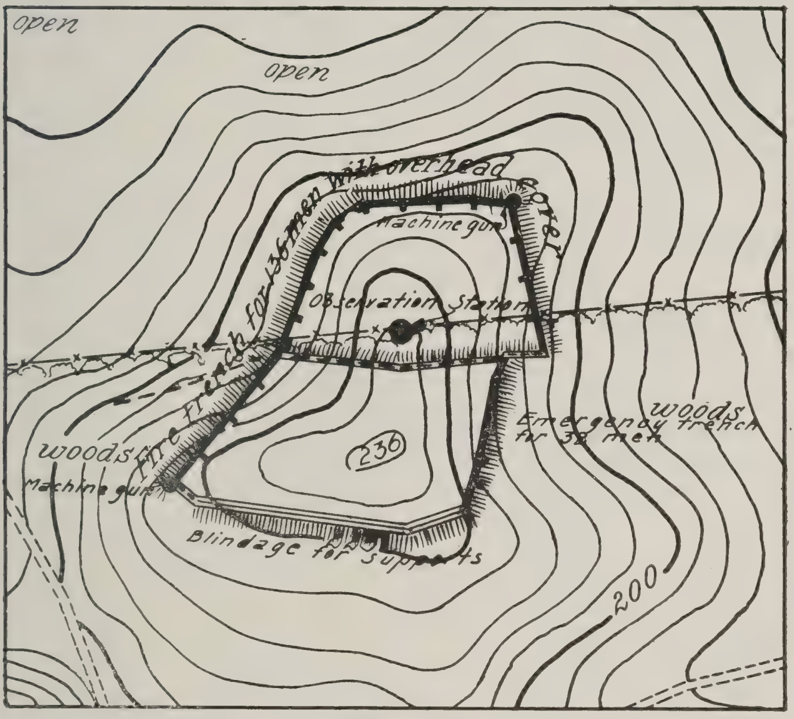


Fig. C.

to suit the slope and other local conditions of site, the trenches would be almost useless. The writer does not advocate the memorizing of all the dimensions of his eighteen or nineteen different profiles, but, for all who wish to master the art of fortification, he does advocate the study of *the proper modifications of standard profiles to suit local conditions of site*. It is believed that the character of these modifications is little realized, except by those who have actually tried to construct them on varied ground. Witness the following remarkable statement which appears in one of the above discussions: "If the front slopes are actually 19 to 23 degrees, as the author states, *this is steeper than is desirable for fire trenches.*"

This is undoubtedly true for those who are looking for a *site to suit the normal profile* instead of *a profile to suit the site.* I believe the Russians found the slopes of 203-Meter Hill, which were more than 23 degrees, quite "desirable for fire trenches."

The writer does not remember having seen any previous article dealing with the proper modifications of standard profiles to suit local conditions of site, and because he has gone to the trouble to work out all the details which must be known by anyone in charge of the actual construction of his trenches he is quite naturally, though I believe unjustly, criticised for complexity. For how can the knowledge of these modifications be gained unless actual modified profiles are drawn to suit different conditions?

My idea is that with the rough plan, Fig. C, and standard profiles the working parties would be started, but the officer in charge must have such a knowledge of the proper modifications of these profiles, and other details of construction, that when completed the work will be properly adapted to the site, and drawings such as those submitted with my original article are not at all too complicated to properly represent this finished product.

INFLUENCE OF THE CLOSED WORK.

It is said that my plans show the influence of the redoubt, or old pattern of closed work. I am well aware that the principle appears to be fully established that the modern supporting point should consist of a group of simple rifle trenches, so located as to bring fire over all probable lines of attack, with cover for supports nearby connected by the necessary communicating trenches. If, however, these trenches,*due to the configuration of the ground,* naturally lie on the perimeter of a closed work, as happened in my plans, and in both of the solutions submitted with the above discussions, is there any inherent defect in the closed work which would require us to shift our trenches to more disadvantageous positions in order to avoid a closed perimeter?

LOCATION OF THREE-INCH BATTERY.

In my original article, on page 431, it is stated that "the whole group covers an almost ideal location for a 3-inch battery used as a 'stabbing' battery, * * *." Such use of field artillery is well recognized (see p. 51, "dagger batteries," Notes on Field Fortification, Army Field Engineer School, Fort Leavenworth, 1912; and "Some Recent Tendencies in Field Engineering," by Capt. Holt Wilson, D. S. O., R. E., PROFESSIONAL MEMOIRS, p. 663, Vol. IV). But due to my failure to *define the term,* and state that it was only to be used in the final stages of the attack, it was apparently assumed that I favored such a position for *all my artillery, all the time.* Such an assumption was as unfortunate as it was erroneous, and my critics who refer to my work as being "crowded with infantry and artillery" simply show their complete ignorance of the term "stabbing battery."


Approximate Dead Area
From Upper Trench...
From Lower Trench...
Scale of Yards
100 50 0 100 200 300 400 500
Lower Trench
Upper Trench
Line of Fence & Edge of Woods

Fig. D.

MAJOR HART'S SOLUTION.

In connection with Major Hart's excellent solution of this problem, it appears to the writer that he has gained the following *advantages*

a. More men firing to the front;
b. Greater dispersion of target;
at the expense of the following disadvantages:
a. Greater length of trench to dig;
b. Poorer communications;
c. Supports at a greater distance from the firing line;
d. Loss of considerable field of fire due to lower position on the slope, and without changing the following points:
a. Trenches still form practically a closed work;
b. Ratio of depth to width of work has remained unchanged;
c. Concealment of work about the same.

The question then is: Does this increase in *frontal fire* and *dispersion* of trenches compensate for the *increased labor, poorer communications* with supports and *restricted field of fire?*

To appreciate these disadvantages, let us compare his solution with a similar rough sketch of my work (Fig. C).

a. To complete the trenches shown, it would require roughly:

	Major Harts.	Fig. C.
Fire trench, overhead cover	190 linear yards	159 linear yards
Emergency trench	90 linear yards	40 linear yards
Cover for supports	83 linear yards	60 linear yards
Communicating trenches	70 linear yards	70 linear yards
Total	433 linear yards	329 linear yards

An increase of about 32 per cent in labor of construction.

b. Supports must travel *200 yards* through the trenches, instead of about *100 yards* to reach the center of the front firing trench.

c. Supports are 133 yards in rear of firing line, instead of 78 yards.

d. The field of fire to the *front* from the *lower trench* is entirely restricted to less than *450 yards,* while on the left *front* the areas covered by fire beyond 200 yards are very small

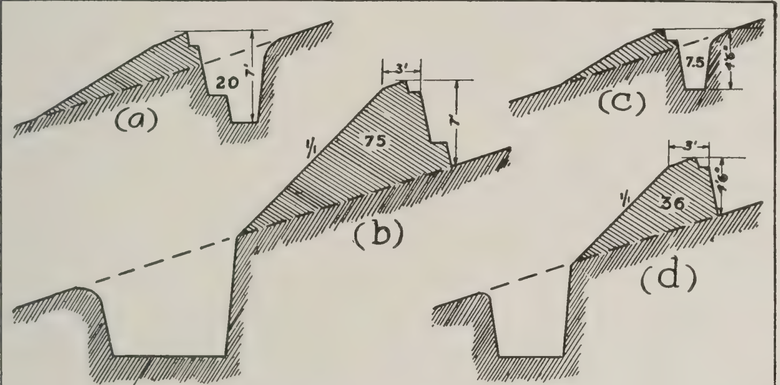
I am afraid I can not agree with the statement made in one of the discussions that "the question of whether or not a given trench is 10 yards too high or too low on a hill is of insignificant importance, * * *." To me, in this case, it seems of paramount importance.

A comparison of the *dead areas* from the upper and lower trenches is shown in Fig. D. This is not submitted as an absolutely accurate chart, for it was surveyed with transit and stadia only within the 450-yard range; beyond that it was estimated, by ob-

serving the hill from different points in the foreground, but is believed to be substantially correct.

In connection with this the illustrations, Figs. 4 and 5, page 439, taken looking to the left front from the upper and lower positions, respectively, show clearly the broken character of the foreground, which affords so much cover from fire from the lower trench.

In view of the above facts, it is my opinion that the advantage of the greater number of rifles pointing to the front from the lower trench is more than offset by the decrease in the field of fire from that location. If greater frontal fire than is given by the trenches in my original plan, Plate II, is considered necessary, and this appears to be the consensus of opinion, I would prefer to swing the trench from B to A, Plate II, around parallel to the contour into



Comparison of excavated trenches on slope of 17° with parapet above the ground as suggested by Maj. Connor for positions in the woods.

Fig. E.

some such position as that shown by the *broken line* in Fig. C. This, I believe, would have the desired effect, without sacrificing the command of the foreground so necessary at this site.

As for the dispersion of trenches, which is recognized to-day as the most effective method of protection against artillery fire, there must be some limit to this dispersion. We must not forget that with this desirable dispersion we usually get very undesirable increased labor and decreased facilities for communication.

MAJOR CONNOR'S SOLUTION.

The works designed by Major Connor would certainly be almost impregnable, but as designed it seems to involve at least three or four times the labor involved in the original plan.

This extra labor is added in three ways:

1st. In constructing 342 more linear yards of trench, an increase of almost 100 per cent;

2d. In the excavation of large rooms as bombproofs, involving at least three times as much labor as that required for the cover shown in Fig. 6, Plate IV;

3d. In constructing parapets above the ground for all cover within the woods. This increases the labor from 200 to 500 per cent over that required for trenches giving the same amount of cover.

In connection with this suggestion to eliminate water in the trenches, by constructing parapets above the ground for all those located in the woods where concealment is unnecessary, I have prepared Fig. E, which shows the comparative amounts of excavation necessary on a slope of 17 degrees, such as that occupied by trench AB, Plate II.

Fig. C, "a," shows profile of complete trench, with 7-foot cover; as shown in Fig. 4, Plate IV, *area to be excavated equal 20 square feet.*

Fig. C, "b," shows parapet above ground furnishing same amount of cover as "a;" *area to be excavated equal 75 square feet.*

Fig. C, "c," shows profile of simple standing trench, with 4-foot 6-inch cover; *area to be excavated equal 7.5 square feet.*

Fig. C, "d," shows parapet above the ground furnishing same amount of cover as "c;" *area to be excavated equal 36 square feet.*

It is believed that if proper steps are taken to locate the trenches so that they all slope towards predetermined points at which drainage ditches are located as outlets, and if surface ditches are placed above the trenches as shown in all profiles, Plate IV, and in plan on Plate II, there will be no trouble on account of water in the trenches, and all the above excessive excavation will be unnecessary.

MACHINE-GUN POSITIONS.

There seems to be some question as to the proper use and location of machine guns. Some advocate holding them all with the reserve, and not making any preparation for them in the trenches, while others not only assign them to the defense of different sectors, but also prepare many spare emplacements for each gun, thus multiplying their efficiency. Personally, the writer favors the latter view, and it is thought these spare emplacements should be placed where the gun has the best field of fire, and not withdrawn for safety if this results in a much more restricted field of fire.

It is not seen why the machine guns are "immobilized," merely because preparations are made to use them when in the trenches.

OBSERVATION STATIONS.

The question of the proper design of officers' observation stations is religiously overlooked in most modern articles of field fortification. Yet the officers are present and must properly control and direct the fire. They have no rifles, so would be decidedly in the way in the firing trenches. Where will they go, and what pro-

tection may they expect in a field work? Perhaps a hole in the ground in rear of the firing trench, as used in the recent Balkan War, is sufficient for the company officers. Perhaps Major Harts' suggestion of a platform in a tree on top of the most prominent point on the hill would do for our battalion commanders. It seems to me, however, that he has great faith in the invulnerability of majors, if he expects one to last long in such a position.

I believe that if two days are available for preparation, some provision should be made for the protection of certain observing points, not only for the commanding officer but also for each company commander. The question then is, What is the proper design, to give adequate facilities with the least amount of labor? We certainly will not arrive at a solution by always avoiding the problem.

Three different types of observing points were included in my plans, all original and each adapted to suit its site. The immediate result of this originality was to raise the cry of "needless complexity." However, aside from the question of complexity of drawings, I would like to see some discussion of how to accommodate the officers so that they can best control the course of the action.

OBSTACLES.

It is not thought that in a work of this character there will be time and labor available for protecting the obstacle with a parapet as suggested by some. Furthermore, such a parapet, instead of concealing the obstacle, would render it more visible, due to the shadows cast by the vertical slopes.

A straight line is favored for the obstacle, as it is easier to lay out and can be more effectively enfiladed from neighboring trenches. Of course, in construction this may be deviated from and every advantage taken of natural features of the ground for concealment, keeping it under fire from the trenches at a distance of not less than 50 yards.

MAPS.

With reference to the reported discrepancies in the maps, the writer was somewhat surprised at this criticism, but after careful investigation disagrees with his critics as to the facts. The front slope, where the firing trench crosses YZ, is shown on Plate II as $23° 30'$; on Plate III as $22° 47'$; and is stated in the text as "from $19°$ to $23°$." This seems to show that Plates II and III are in remarkable accord with each other and with the statements in the text, instead of being "graphical representations, no two of which agree, and all of which are at variance with the statement made in the text."

CONCLUSION.

The writer would like to thank all those who have shown their interest in his study of this problem by submitting their views and suggestions in the above discussions. He was particularly pleased to receive the views of several officers not of the Engineers, the discussion submitted by Lieutenant-Colonel Dugan and Major Cabell

of the Cavalry being especially valuable, as they covered all points touched on in the original article, either approving or disapproving each phase of the subject. He trusts that no one will deem him presumptuous in presenting in these comments his own views, even though they differ in some cases rather widely from those of officers of much greater rank and experience.

This study has made the writer realize more than ever before the truth of that statement with which Colonel Kuhn concludes his discussion: "Rarely will it happen that actual topographical features permit of the complete realization of theoretical principles. It is in the adaptation to actual ground forms that one learns the limitations of fortifications, as well as their possibilities."

The Gladstone Dock, Liverpool

The illustration shown above is reproduced from a photograph recently taken of the Gladstone dock, and shows the progress which has been made with this great work. The view is taken from the temporary embankment thrown across the river front. As our readers are already aware, the dock is intended to be used when necessary as a dry dock, and will be 1,020 feet long—or 137 feet 6 inches longer than the *Olympic*—and the entrance will be 120 feet wide. The sill is laid at a level of 25 feet below the old dock sill. The engraving shows that the walls are complete. The work is being carried out with all possible dispatch by the Mersey Docks and Harbour Board, and it is hoped to have the dock ready for service next year.—*The Engineer,* London.

Test of Anchor Bolts at Keokuk, Iowa

BY

Maj. CHARLES KELLER
Corps of Engineers; Member American Society Civil Engineers

To determine the efficiency of the anchor bolts of the miter sill of the new lock at Keokuk, six extra bolts were provided and tested. For these test bolts, holes, 4 inches in diameter and 8 feet deep, were drilled in the limestone of the river bed inside the lock cofferdam about 75 feet downstream from the miter sill, and in each hole a 2-inch bolt, on which four barbs, 1 inch long and 3/8-inch deep, had been cut as shown, was inserted and firmly "fox-wedged" in place. At a depth of about 6 feet a thin water bearing seam had been encountered in the limestone, so that the 1 to 2 grout, with which the drill holes were then filled, was necessarily placed with air under sufficient pressure to keep the water down. The test bolts were permitted to remain in place for nearly a year before subjecting them to the test now described.

In October, 1912, these six bolts were tested to destruction by means of the apparatus fully shown on the accompanying drawing.

One of the six bolts was tested with a double nut, and the remainder with a single nut. In no case did the threads strip. A single nut is evidently sufficient to develop the full strength of the bolt as made and used.

In general, it may be stated that the bolts began to stretch rapidly when the pull on the bolts rose to about 45 tons. After this stress was applied, stretching was so rapid that it was only by rapid pumping that the pressure on the jack could be maintained and increased. In three cases rupture occurred at a total stress of about 67½ tons, and all fractures showed the characteristics of good mild steel. With one exception, all broke immediately under the nut, and in the case of the exception the bolt broke at the first barb below the surface.

The anchor bolts in the actual miter sills will be placed in a manner similar to those in the test, except that in the sill itself the bolts will be incased and grouted in a 2½-inch gas pipe, this pipe being

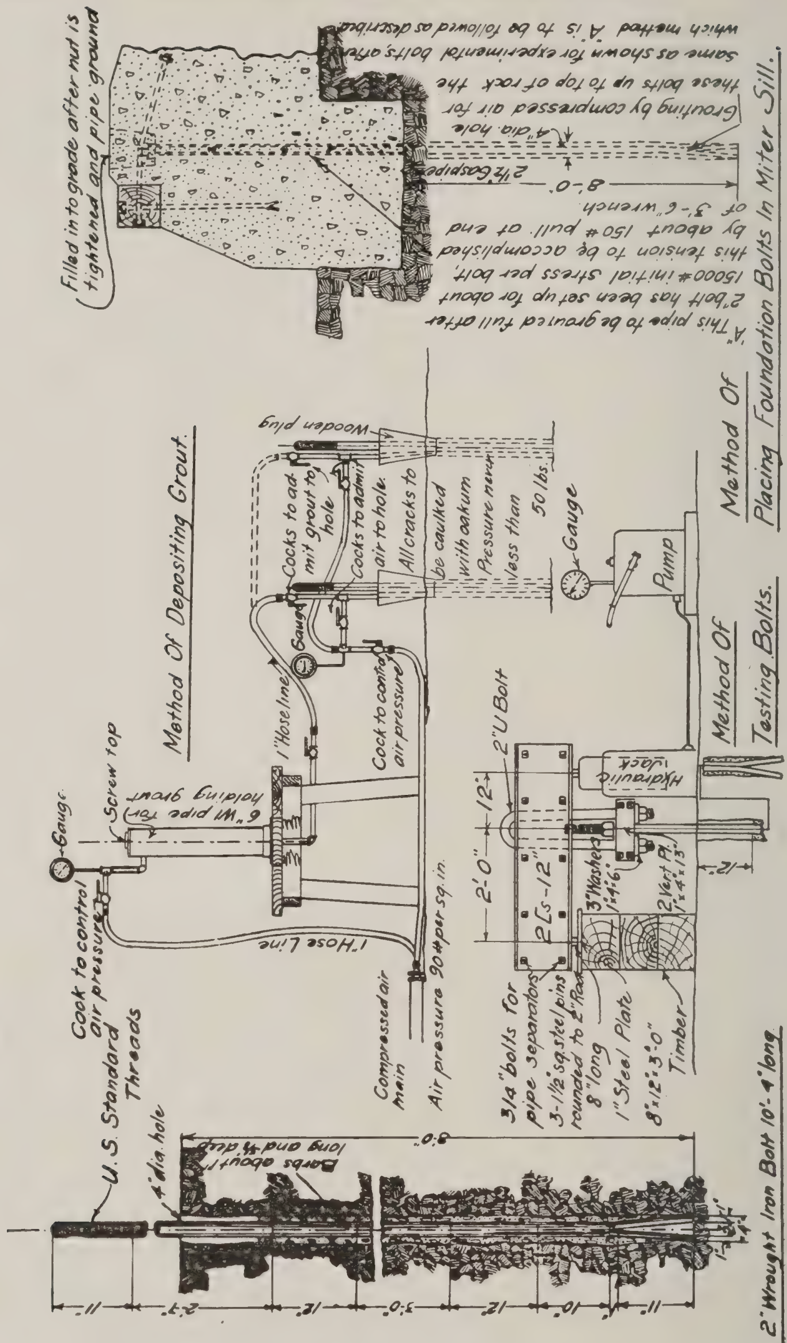
intended to prevent rupture of the concrete sill by the stretch of the bolt.

In order to determine the safety, or otherwise, of the proposed anchor bolts, it is necessary to find the hydrostatic pressure upon them should the water get under the sill. The bolts are spaced 4 feet apart and the sill is 4 feet wide, so that, with a 40-foot head the uplift will be $4\times4\times62.3\times40=39{,}870$ pounds. The weight of the masonry is: $4\times4\times7\times140=15{,}680$ pounds, and the total upward pressure is therefore $39{,}870-15{,}680=24{,}190$ pounds.

The miter-sill anchor bolts are not barbed, so that the single case of the bolt that failed at the first barb (No. 4) may be disregarded. Of the remaining five bolts, three showed an ultimate resistance of 67.5 tons per bolt, and two failed when the load was 60 tons. The maximum upward pressure being about 12 tons, the factor of safety against rupture is about 5, and, assuming the elastic limit of mild steel as approximately two-thirds of its ultimate strength, the factor of safety against permanent distortion is about 2 2-3. This is a minimum, since it is scarcely probable that the bolts will be subjected to the full lifting effect of the hydrostatic head. Evidently, the miter-sill anchorage is amply safe. The table and drawing herewith furnish further details.

No.	Original 2-inch Bolts.				Tension Load Applied.			Broken Bolt.	
	Full size.		Through threads.		First Pull.				
	Diam. in inch.	Area in sq. inch.	Diam. in inch.	Area in sq. inch.	Tons on bolt.	Stress, lbs. per sq. inch.	Total Elongation in ft.	Net diam. in in.	Net area in square inches.
1	2	3.1416	1.64	2.0612	7.5	7280	0.000	1.46	1.6741
					22.5	21830	0.003		
					30.0	29110	0.008		
					37.5	36380	0.008		
					45.0	43660	0.014		
					52.5	50940	0.024		
					60.0	58220	0.079		
						Second Pull			
					7.5	7280	0.000*		
					15.0	14550	0.003		
					22.5	21830	0.005		
					30.0	29110	0.005		
					37.5	36380	0.007		
					45.0	43660	0.010		
					52.5	50940	0.010		
					60.0	58220	0.012		
					67.5	65490	Failed		
						First Pull			
2	2	3.1416	1.62	2.1124	7.5	7100	0.000	1.60	2.0106
					15.0	14200	0.006		
					22.5	21300	0.008		
					30.0	28400	0.010		
					37.5	35500	0.016		
					45.0	42610	0.029		
					52.5	49710	0.045		
					57.0	53970	0.111		
						Second Pull			
					15.0	14200	0.000*		
					22.5	21300	0.003		
					30.0	28400	0.004		
					37.5	35500	0.010		
					45.0	42610	0.010		
					52.5	49710	0.010		
					60.0	56810	0.020		
					67.5	63910	Failed		
3	2	3.1416	1.64	2.0612	7.5	7280	0.000	1.50	1.7672
					15.0	14550	0.000		
					22.5	21830	0.001		
					30.0	29110	0.003		
					37.5	36380	0.007		
					45.0	43660	0.015		
					52.5	50940	0.046		
					55.5	53850	0.077		
						Second Pull			
					7.5	7280	0.000*		
					15.0	14550	0.001		
					22.5	21830	0.005		
					30.0	29110	0.005		
					37.5	36380	0.008		
					45.0	43660	0.010		
					52.5	50940	0.013		
					67.5	65490	Failed		
4	2	3.1416	1.66	2.1642	7.5	6930	0.000		
					15.0	13860	0.003†		
					22.5	20790	0.005		
					30.0	27720	0.005		
					37.5	34660	0.007		
					45.0	41590	0.015		
					52.5	48520	0.038		
					57.0	52680	Failed		
5	2	3.1416	1.62	2.1124	7.5	7100	0.000	1.52	1.8146
					15.0	14200	0.005		
					22.5	21300	0.006*		
					30.0	28400	0.007		
					37.5	35500	0.016		
					45.0	42610	0.028		
					52.5	49710	0.067		
					60.0	56810	Failed		
6	2	3.1416	1.66	2.1642	7.5	6930	0.000	1.48	1.7203
					15.0	13860	0.005		
					22.5	20790	0.005*		
					30.0	27720	0.006		
					37.5	34660	0.012		
					45.0	41590	0.023		
					52.5	48520	0.051		
					60.0	55450	Failed		

*Bolt failed through the threads. † Bolt failed at first barb below the top.


2" Wrought Iron Bolt 10'-4" long
U.S. Standard Threads
4" dia. hole
Barbs about 1" long and ¼" deep
Method Of Depositing Grout.
Cook to control air pressure
Gauge
Screw top
6" W.I. pipe for holding grout
1" Hose Line
1" Hose line
Compressed air main
Air pressure 90# per sq. in.
Cock to control air pressure
Gauge
Cocks to admit grout to hole
Cocks to admit air to hole.
All cracks to be caulked with oakum
Pressure never less than 50 lbs.
Wooden plug
Method Of Testing Bolts.
2" U Bolt
2 [s-12"
3/4" bolts for pipe separators
3-1½" sq. steel pins rounded to 2" Rad. 8" long
1" Steel Plate
8"x12"x3'-0" Timber
3" Washers 1"x4"x6"
2 Vert. Pl. 1"x4"x13"
Hydraulic Jack
Gauge
Pump
Method Of Placing Foundation Bolts In Miter Sill.
Filled in to grade after nut is tightened and pipe ground
2½" Gaspipe
4" dia. hole
8'-0"
Grouting by compressed air for these bolts up to top of rock the same as shown for experimental bolts, after which method "A" is to be followed as described.
"A" This pipe to be grouted full after 2" bolt has been set up for about 15000# initial stress per bolt, this tension to be accomplished by about 150# pull at end of 3'-6" wrench.

Gen. George Gordon Meade

Fifty years ago to-day (June 28, 1913), Major-General George Gordon Meade, a former officer of the Corps of Engineers, United States Army, was placed in command of the Army of the Potomac. To this Army was then entrusted, as it had been for nearly two years, the defense of the city of Washington and the defeat of the able General Robert E. Lee, another former officer of the Corps of Engineers, commanding the Confederate Army of Northern Virginia. While these armies were nearly equal in numbers, equipment, and supplies, there must be added to the Army of Northern Virginia the confidence inspired by a series of almost unbroken victories over the Army of the Potomac.

General Meade had served with the Army of the Potomac throughout nearly the entire two years of the Civil War preceding the Battle of Gettysburg and had proven himself a courageous, steadfast, and reliable general under all conditions. He had thus risen to be second in rank in that Army to General Hooker who commanded it at the beginning of the Gettysburg campaign. When President Lincoln became dissatisfied with General Hooker and relieved him from command upon his rather hasty request to be so relieved, the choice of a commanding general naturally fell to Meade. It was a most desperate and trying time for any officer, but Meade, with deep appreciation of the responsibility thrust upon him, took command of the Army in a spirit which, while realizing the tremendous responsibility, was determined to go ahead with all his powers.

On this the 1st day of July, 1913, just fifty years after the beginning of the Battle of Gettysburg, and while the Blue and the Gray who then fought together in death-like grip are meeting and clasping hands with mutual regard and admiration in a great semi-centennial celebration of that battle, it seems especially fitting to append a short sketch of the life of the Commanding General of the Union Forces in the PROFESSIONAL MEMOIRS, published by the Corps of Engineers—the Corps in which he served conspicuously

for many years before the Civil War and to which he brought great honor by his conduct during that war.

General George Gordon Meade was born December 31, 1815, in the city of Cadiz, Spain, where his parents, who were citizens of the United States, were temporarily residing. His father was not only in business of his own but held an appointment as Navy Agent for the United States. George Gordon Meade was the eighth child in a family of ten and was five years old when his father returned to the United States. He studied at first in a private school in Philadelphia and later at boarding school at Mount Airy, a few miles outside of Philadelphia. One of the principals of this school was a graduate of the Military Academy at West Point, in consequence of which the school used West Point as a model, and the boys received instruction in company drill, sentry duty, and the like. At Washington he was for a short time a pupil of Salmon P. Chase, later Secretary of the Treasury under President Lincoln. General Meade seems to have had at an early age a strong liking for mathematics and kindred studies and desired to become a civil engineer. It not being practical for him to get the necessary education, he obtained an appointment to West Point in 1831 when he was but 15 years and 8 months old.

He graduated four years later No. 19 in a class of fifty-six. During his graduation leave-of-absence he took an appointment as an assistant on the survey of the Long Island Railroad, having in mind even at that time that he should later resign and enter railroad construction work. He was assigned to the Artillery, in which he served until October 26, 1836, on which date he resigned to accept employment in civil life. He was employed as an assistant engineer in railroad work and also later on the survey of the mouth of the Sabine River, at that time the boundary line between the United States and the Republic of Texas. Later, during the period from the latter part of 1837 until May 19, 1842, he served as assistant on the survey of the delta of the Mississippi River, the astronomical work on the survey for determining and marking the boundary line between the United States and Texas, and as assistant on the survey of the Northeastern Boundary between the United States and the British provinces. Finding this work irregular and seeing no opportunity for study and profitable employment as a civil engineer, he accepted an appointment on May 19, 1842, as second lieutenant in the Corps of Topographical Engineers, and from that date until November, 1843, continued on the survey of

the Northeastern Boundary Line above referred to. He was then assigned to duty in the construction of light-houses and in surveys of Delaware Bay. On the 12th of August, 1845, he was ordered to join a force at Fort Jessup, Louisiana, under command of Gen. Zachary Taylor with a prospect of engaging in a war with Mexico. In this war he served with distinction on the staffs of General Taylor and General Scott in the battles of Palo Alto, Resaca de la Palma, and Monterey, being brevetted for conspicuous service in the latter battle. He was promoted to first lieutenant, Corps of Engineers, in 1851, and to captain in 1856. Following the close of the Mexican War, he was engaged for a time upon light-house construction, but during the four years preceding the Civil War he had charge of the survey of the Great Lakes. In August, 1861, following the breaking out of the Civil War, he was put in command of the Second Brigade of the Pennsylvania Reserve Corps, the latter forming a division in the Army of the Potomac, with which Army he served throughout the war. In the Peninsular Campaign in the spring of 1862 he took part in the battles of Mechanicsville, June 26; Gaines Mill, June 27; and the Battle of Glendale, June 30, in which he was severely wounded. Later in the year he was engaged in the Battle of Manassas and during Lee's first invasion of the North took part in the battles of South Mountain and Antietam, a part of which time he was temporarily in command of a corps. Still later in 1862 he took a most conspicuous part in the Battle of Fredericksburg and in the spring of 1863 in the Battle of Chancellorsville, in which he commanded the Fifth Corps. He was still commanding this corps June 28, 1863, when placed in command of the Army of the Potomac.

It is thus evident that he had been in a position to obtain as accurate a knowledge of the fighting abilities and strategy of the generals and men composing the Confederate Army of Northern Virginia as any man in the Union Army. It is doubtful if, without this experience, he would have been able to command the Army so successfully as he did in the great Battle of Gettysburg, which began only three days after he was placed in command. As a result of his victory at Gettysburg he was promoted to be a brigadier-general in the Regular Army, July 3, 1863, though he remained a major-general of volunteers in command of the Army of the Potomac until relieved by General Grant early in 1864. Even then he continued to be the senior officer in command under General Grant throughout the Wilderness Campaign and the Siege of

Petersburg up to the final capitulation of Lee and the Army of Northern Virginia at Appomattox Court House. To enumerate all the battles of this campaign in which he took part, always creditably and many times most conspicuously, would be to make a practically complete enumeration of the battles in which the Army of the Potomac took part in 1864-1865.

On July 1, 1865, following the close of the Civil War, he was placed in command of the Military Division of the Atlantic which included the Atlantic States as far south as South Carolina, a position he held with the exception of a very short period until his death in Philadelphia, November 6, 1872. During this period he also served on many boards, being president of the board for making recommendations for brevets to the grade of general officers and also of a board for retiring disabled officers. Naturally, after a terrible four years' war these two boards were of tremendous importance.

In addition to his eminent standing in the military world, he was well known for his scientific attainments. In 1863 he was made a member of the Historical Society of Pennsylvania and, in 1865, an LL. D. of Harvard and a member of the Philadelphia Academy of Natural Sciences.

General Meade's life was marked, not so much by any great brilliancy as by steadfastness of purpose coupled always with a determination to do the very best in his power and to shirk no task assigned, no matter how great the responsibility.—A. A. F.

Errata in This Number

Page 473, 19th line from the top, change the word "not" to "now." Same page, 5th line from the bottom, replace the words "it is" by "if."

Page 477, 16th line from the top, add the word "have" after "and." Same page, 20th line, add the word "a" after "make." 23d line, same page, add the word "out" to "lays."

Page 479, 20th line from the top, change the word "involved" to "involves."

Book Review

A Critical Study of the German Tactics and of the New German Regulations. By Major De Pardieu of the French Army; translated by Capt. Charles F. Martin, Third U. S. Cavalry. Published by the U. S. Cavalry Association, Fort Leavenworth, Kansas, 1912. Price, $1.00, prepaid.

This is one of those rare books that gives both sides of a question by one who has made a deep study of that question.

German tactics for all arms have been revised, in some cases most radically, since the close of the Russo-Japanese War in which the ideas of German military students were so successfully carried out by the Japanese.

As stated by the translator in his preface, Major Pardieu was chief of staff to the military governor of Dunquerque when his study was published.

As a measure of its true worth, it is noted that immediately after its publication in France, it was translated into both Russian and German.

In this work Major Pardieu takes up first a brief study of the infantry regulations, followed by similar studies concerning the field artillery and cavalry.

The Germans have recently adopted both light and heavy howitzers and to these Major Pardieu gives particular attention and discusses the use of artillery, both heavy and light, in a way highly useful to all officers, and especially artillery officers, engineers, and others who may have to take part in choosing artillery positions or in the construction of trenches and other works of defense against hostile artillery.

These three subjects take up only a little more than one-third of the book, the remainder being given to a study of the battle from both the offensive and the defensive sides, followed by a discussion of the preparatory combat and the final outcome of the battle—whether victory or defeat.

Then follows a study of tactics in general, both from the offensive and defensive sides, the book being closed by a brief but keen

strategical synopsis of the manner in which a campaign would probably be carried out by the German Army.

In all of these discussions Major Pardieu sets forth clearly the French point of view, acknowledging frankly what is good in the German tactics and suitable for adoption in the French Army as well as frankly condemning what he thinks an error in either case. The author is a keen student of the German character as well as that of the French, and this fact alone makes his work vastly more valuable than any similar work of its size known to the writer of this review.

A careful reading of this book will impress upon one the necessity of knowing the enemy as thoroughly as one knows his own army, if he hopes to get the best results from the use of the latter. This knowledge must cover not only the enemy's arms and methods of supply and the skill with which he uses his weapons, but must include a knowledge of his habits of thought and general mental processes—all to be combined with the best possible knowledge of his probable plan of campaign.

It is believed that this book will be of great value to all officers of the army and particularly so to all engineer officers who may wish to be able, should the occasion arise, to take a conspicuous part in any struggle with a foreign power.—A. A. F.

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Compiled by Henry E. Haferkorn, Librarian, Engineer School.

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(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
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(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
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(21) Proceedings, American Society of Civil Engineers.
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(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(7?) Institution of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review. London.
(80) Journal, American Society of Engineering Contractors, N. Y.
(82) Journal, New England Water Works Association, Boston.
(83) National Waterways, Washington, D. C.

*Now combined under title: Concrete-Cement Age.

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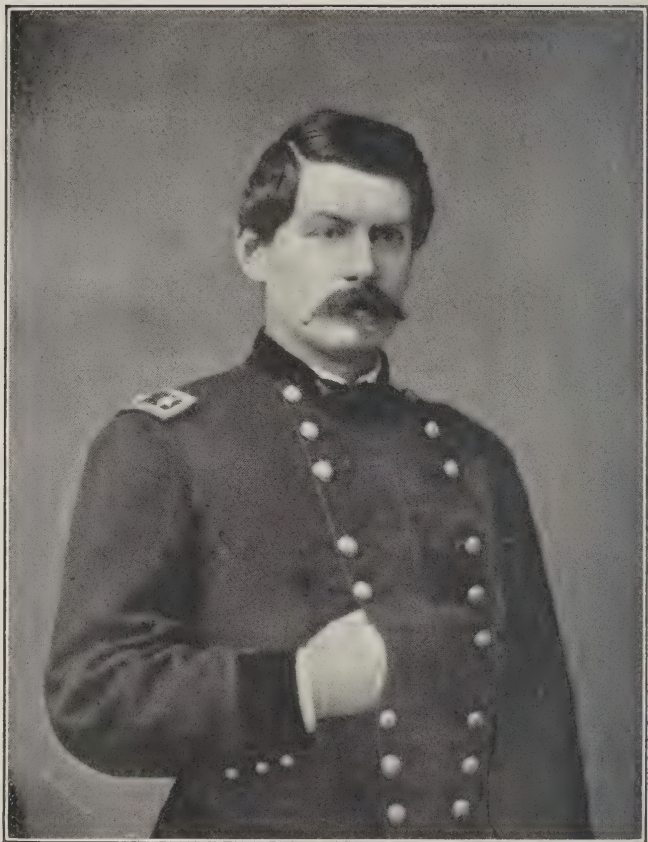

GEN. GEORGE BRINTON McCLELLAN

BORN 1826—DIED 1885

PHOTOGRAPH TAKEN 1862

SEE P. 607

Rebuilding Jetties at Humboldt Bay, California

BY

Mr. Morton L. Tower*
Assistant Engineer; Member American Society of Civil Engineers

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Humboldt Bay, on the northern coast of California, latitude 40° 45′, is the principal lumber shipping port of the State of California, being in the center of one of the most productive forest areas known to man.

The entrance to the harbor is between two low sand spits. The area of the bay at low water is 11.4 square miles and at the contour 6 feet above low water it is 25.3 square miles. The mean rise and fall of the tide is 4.3 feet with extreme ranges as great as 11 feet, from 9½ feet above to 1½ feet below the plane of reference established by the United States Coast and Geodetic Survey at the mean of the lower low waters.

In its natural condition the erosion of the tidal currents, modified by the ever present but constantly changing surf conditions, resulted in a channel unstable as to position, depth, or direction, with a general ruling depth of about 12 feet at low water.

The harbor was first examined with a view to improving the condition of the entrance in 1877, but, owing to the difficulties to be encountered and the then undeveloped stage of jetty construction methods, no recommendations for actual construction were made. The report stated that a safe entrance could be obtained by the construction of two parallel jetties about 500 yards apart.

Following an appropriation in 1881, control of the South Spit was attempted by the use of brush mattresses. The work being carried on with no appreciable result until 1887, when, following the development of construction methods at Galveston, Tex., and on the Oregon Coast at Yaquina Bay and at the mouth of the Columbia River, a plan for improving the channel conditions by means of two jetties of rip-rap stone deposited from pile trestles

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*Resident Engineer in local charge of work.

was adopted. Work on the jetties was commenced in 1887 and continued until 1899. A total of over 1,000,000 tons of stone was used, the cost of the work being $2,040,203.35.

The channel, both as regards alignment and depth, secured by the improvement was all that was expected and was maintained for several years at not less than 26 feet at mean lower low water with considerable periods when a ruling depth of 30 feet prevailed.

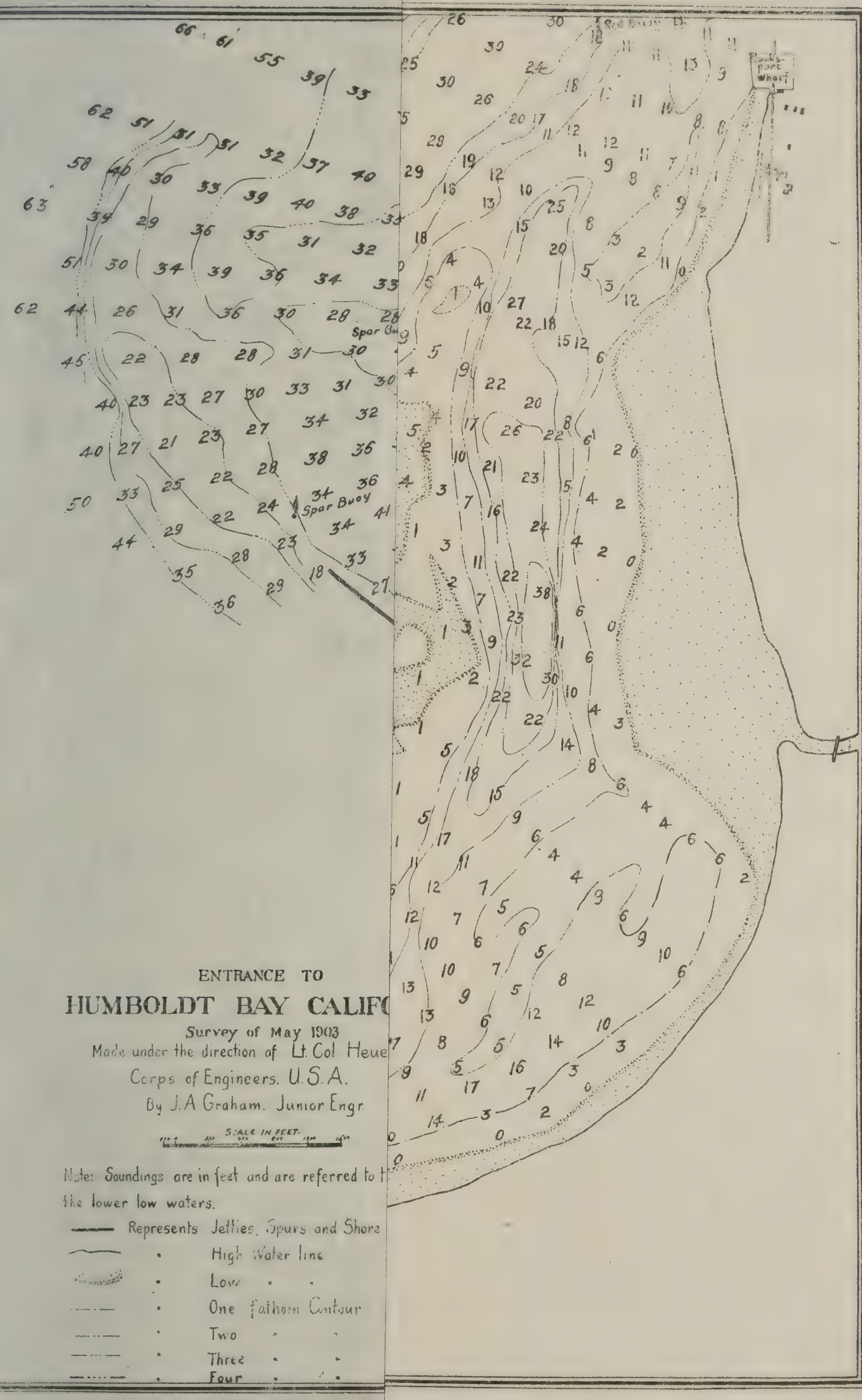
The jetties were constructed by depositing stone ranging in size up to pieces of 10 tons, from pile trestles; the method generally used on the North Pacific Coast.

The effect of the severe surf on these jetties has been to cause subsiding of the outer ends of the work, principally by reduction of slopes and by displacing the top stone. The smaller pieces of stone have been washed away and some disintegration of the stone has occurred. The mass has also settled into the bottom to some extent. Attrition by the sand-laden water is a source of possible loss, considering the total amount of surface exposed.

The subsiding of the jetties allowed an increasing amount of sand to enter the channel from the accumulated deposit in the angles between the shore lines and the jetties. Owing to direction of approach of the flood tide, which coincides with the shoreward moving waves, across the larger area exposed on the southerly side of the entrance, the larger quantity of sand was received and the greatest damage to the channel occurred from that direction.

With the subsiding of the jetties, the condition of the entrance channel deteriorated both in direction and depth. By 1907, the outer ends of the jetties were completely buried in sand at depths of from 14 to 18 feet. The sands carried over the jetty on flood tide being returned with the ebb current and deposited when that current is dissipated on entering the ocean, had formed a narrow continuation of the south jetty, curving northerly and crossing the line of the north jetty prolonged. This condition produced a crooked channel, difficult in all weathers and impossible in foggy and rough weather.

The narrowest and shoalest portion of the channel was generally in the vicinity of the north jetty, and at times was reduced to a width of about 500 feet and a depth of about 16 feet at mean lower low water. The channel constantly changed. During storms from the south the shoal would be forced toward the north and inshore, so that outgoing vessels would head shoreward for a considerable distance in order to reach deep water; with even a moderate sea


ENTRANCE TO
HUMBOLDT BAY CALIF
Survey of May 1903
Made under the direction of Lt. Col Heue
Corps of Engineers. U.S.A.
By J.A Graham. Junior Engr.
SCALE IN FEET
Note: Soundings are in feet and are referred to t
the lower low waters.
Represents Jetties, Spurs and Shore
High Water line
Low
One fathom Contour
Two
Three
Four
Spar Buoy
Spar Buoy
Bucksport Wharf

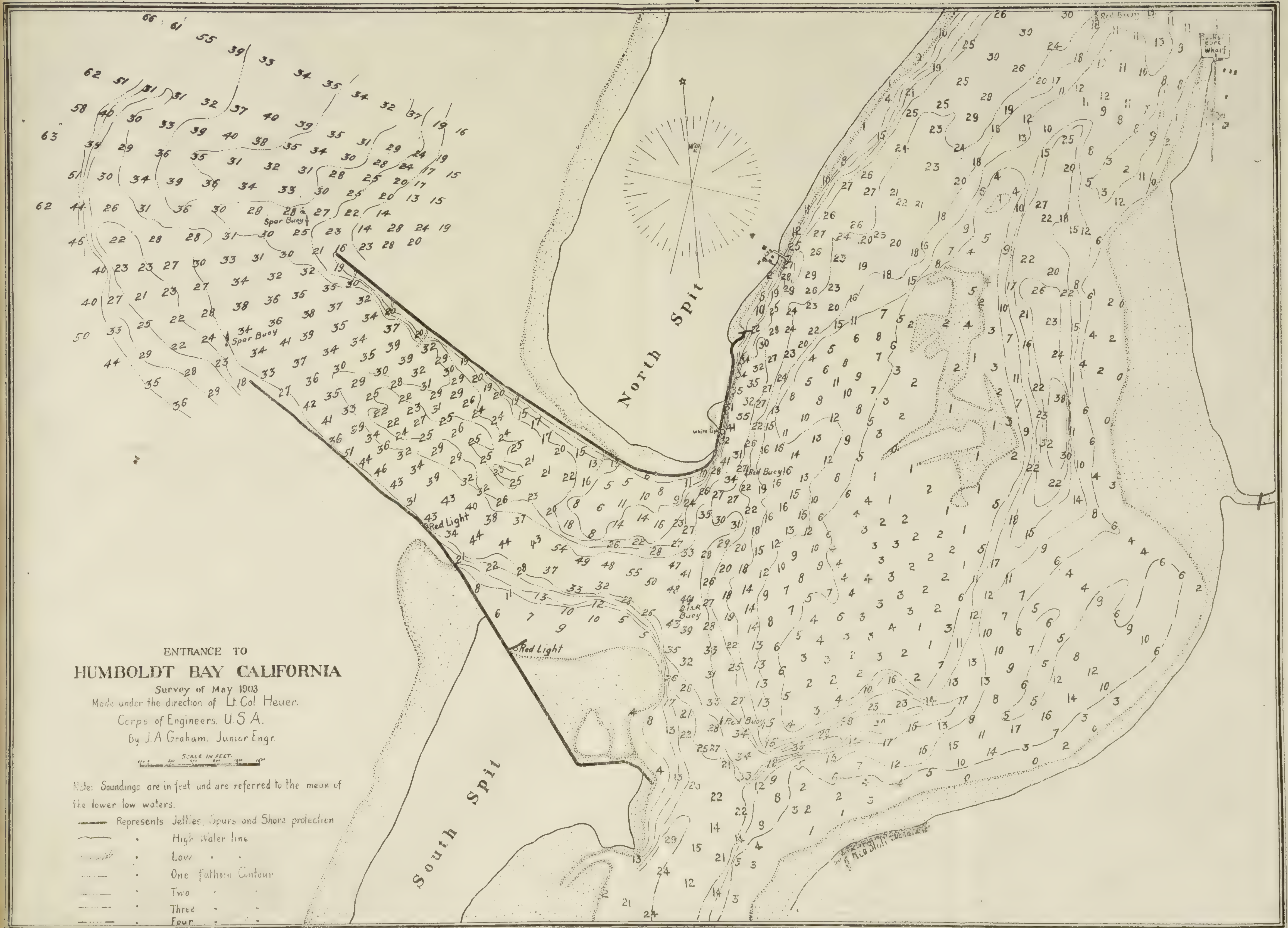

ENTRANCE TO
HUMBOLDT BAY CALIFORNIA
Survey of May 1903
Made under the direction of Lt. Col Heuer.
Corps of Engineers. U.S.A.
By J. A Graham. Junior Engr
SCALE IN FEET
Note: Soundings are in feet and are referred to the mean of the lower low waters.
Represents Jetties, Spurs and Shore protection
High Water line
Low
One fathom Contour
Two
Three
Four
North Spit
South Spit
Spar Buoy
Spar Buoy
Red Light
Red Light
Red Buoy
Red Buoy
White Lt.

Fig. 1.

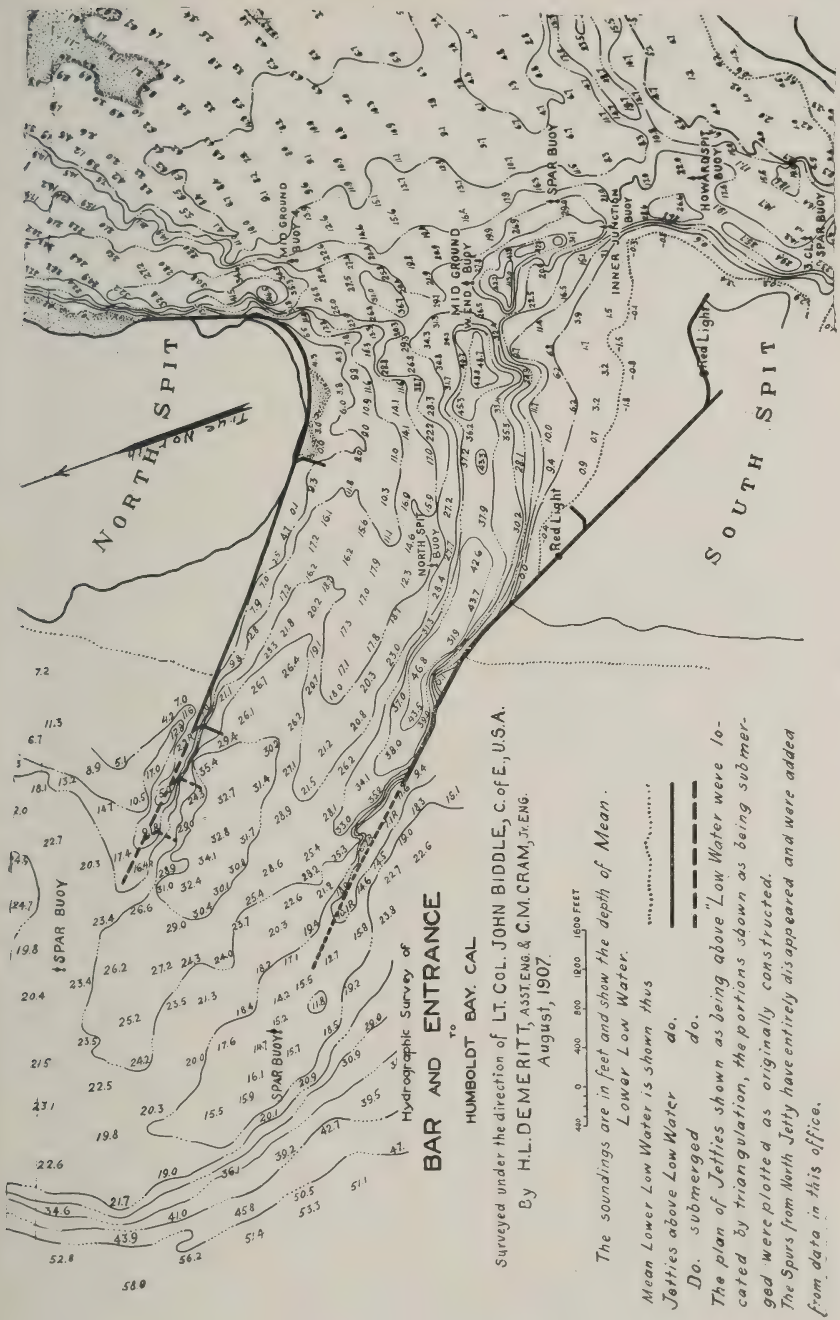

Hydrographic Survey of
BAR AND ENTRANCE
TO
HUMBOLDT BAY, CAL
Surveyed under the direction of LT. COL. JOHN BIDDLE, C. of E., U.S.A.
By H.L. DEMERITT, ASST. ENG. & C.M. CRAM, Jr. ENG.
August, 1907.
400 0 400 800 1200 1600 FEET
The soundings are in feet and show the depth of Mean Lower Low Water.
Mean Lower Low Water is shown thus
Jetties above Low Water do.
Do. submerged do.
The plan of Jetties shown as being above "Low Water were located by triangulation, the portions shown as being submerged were plotted as originally constructed.
The Spurs from North Jetty have entirely disappeared and were added from data in this office.
NORTH SPIT
SOUTH SPIT
True North
Red Light
NORTH SPIT BUOY
MID GROUND BUOY
MID GROUND W. END BUOY
SPAR BUOY
INNER JUNCTION BUOY
HOWARD SPIT BUOY

Fig. 2.

running, the waves breaking on the outer shoal would carry across the narrow channel, thus rendering the passage more difficult by obliterating visible evidence of deep water.

Further danger was encountered in the form of mounds of sand constantly changing in shape, size, and position. In order to be familiar with the channel, constant observation by local pilots was necessary and even regular coastwise vessels could not safely attempt the channel without service from local pilots. The channel reached its worst condition during the fall and winter of 1911.

Fig. 1, Survey 1903 (opposite page 500), shows the most favorable condition of the channel as it existed from the completion of the jetties in 1899 to and beyond the date of survey.

Fig. 2, Survey 1907 (page 501), shows the progressive growth of the South Spit with an isolated 18-foot contour on the northerly side of the channel.

Fig. 3, Survey 1911 (page 503), shows the South Spit extended with but slight depression across the entrance. During the fall of 1911 this shoal increased in length and the depth decreased until the channel assumed its worst position and condition since the completion of the jetties in 1899.

Active work reconstructing the south jetty was commenced in June, 1911. The first stone was deposited in March, 1912. By June, 1912, the south jetty had been extended as shown on Fig. 4 (opposite page 504), 1,500 feet from the high water line.

By the subsiding of the old jetty enrockment there had been left a low section close to the high water shore line (Station 40 to Station 44), where the rock was only a little above mean tide. The considerable quantity of sand forced through this small inlet by the inrush of the flood tide was cut off by the first work in rebuilding the jetty. Fig. 4, Survey 1912, shows the seaward extension of deep water in the channel and the improvement of the channel conditions as to direction and depth.

As these conditions continued to improve during the summer and fall until December, 1912, it is believed that the relief of the channel from the sand moved along the beach and across the shore end of the jetty into the bay was a considerable factor in securing the better conditions.

In September, 1912, about three months earlier than the usual time, there commenced a succession of very heavy waves from the southwest. These waves, entirely independent of the local weather conditions, continued with but very little interruption until Janu-

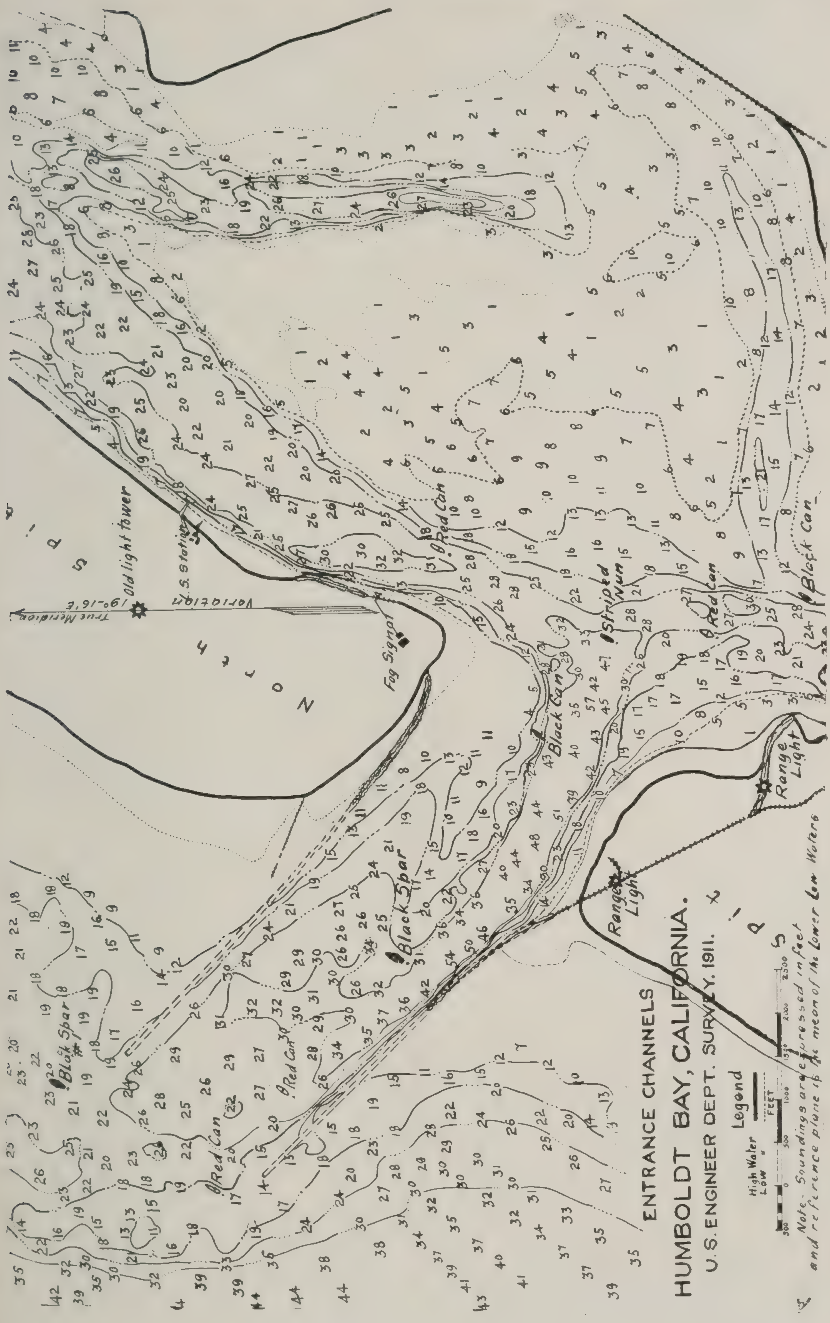

ENTRANCE CHANNELS
HUMBOLDT BAY, CALIFORNIA.
U.S. ENGINEER DEPT. SURVEY. 1911.
Legend
High Water
Low "
FEET
Note: Soundings are expressed in feet and reference plane is the mean of the Lower Low Waters
True Meridian
Variation
19°-16'E
Old light tower
L.S. Station
Fog Signal
Red Can
Black Can
Striped Nun
Black Spar
Range Light
NORTH
SPIT

Fig. 3.

ary 15, 1913. The effect was to throw the accumulated sand near and beyond the outer end of the south jetty across the entrance, causing very difficult navigation conditions for about six weeks. With the cessation of the unusual surf conditions and the commencement of the usual winter weather the channel again assumed the position shown in Fig. 4, or even more favorable conditions, as indicated by information furnished by masters of vessels and local pilots.

The desirability of using large-sized stone is a factor in jetty maintenance which has been well established by experience at all the North Pacific Coast harbors. In planning this work it was decided that the limiting size should be 20 tons. It was also considered desirable that these stones be lowered to place to avoid breaking them or the stone they fell on, which often occurs when stone is deposited by dumping from cars on an elevated track. The use of an unloading crane also permits the placing of stone in a selected position in the jetty, which can not be done when it is dumped from a tramway.

All the quarries adjacent to Humboldt Bay are at a considerable distance from the navigable channels, thus rendering rail transportation essential from the quarry to tide water. The quarry used is 7 miles from the nearest landing.

The largest single item of plant involved in the construction is the cars. It was deemed that it would be cheaper in the end if these were of standard design and hence salable when the work was completed.

A tramway of sufficient strength to carry a 20-ton crane and standard 40-ton railroad equipment is necessarily much heavier and more expensive than the jetty tramways used along the Oregon and Washington coasts, where narrow gauge, special dump cars are used.

The storms of many years had beaten the old enrockment into a compact mass, such that it would have been impossible to drive piles into it, and for a tramway it would have been necessary to place a portion of the piles of each bent in the sand to give it any lateral stability.

The estimated cost of a suitable trestle is as follows:

Six-pile bents 14 feet apart with two lines of 10 by 18 inch stringers under each rail and 6 by 8 inch ties. Rails to be 28 feet above mean lower low water.

Bell Buoy
Red Can
Bucksport
PACIFIC OCEAN
Elk
River
ELS
LIFORNIA
VEY, 1912.
000 FEET
ssed in feet and
ean of Lower Low Water.


ENTRANCE CHANNELS
HUMBOLDT BAY, CALIFORNIA
U.S. ENGINEER DEPT. SURVEY, 1912.
High Water Line
Low " "
FEET
Note: Soundings are expressed in feet and reference plane is the mean of Lower Low Water.
PACIFIC OCEAN
North Spit
South Spit
Bucksport
Elk River
Bell Buoy
Old Light Tower
L.S. Station
Fog Signal
Range Light
Black Spar
Red Can
Striped Nun
Black Can
True Meridian Var. 19°9' E.

Fig. 4.

Material.

6 piles, each 50 feet long, 300 linear feet, at 15 c.	$45.00	
3,500 feet B. M. lumber, at $20 per M. feet	70.00	
170 pounds bolts, nuts and washers, at 3c. per lb.	5.10	
2 rail joints, at $3 each	6.00	
1,120 pounds rails, at 2½ c. per pound	28.00	
107 pounds nails and spikes, at 5c. per pound	5.35	
Material for 14 feet	$159.45	
Material for 1 foot		$11.39
Labor, fuel, and supplies, per day, $59.00.		
Rate of progress (estimated 2 bents, 28 feet), cost per foot		2.11

Plant Required.

1 revolving driver, complete	$20,000	
1 supply and material car	5,000	
3 flat cars, at $600	1,800	
Material yard platforms	5,000	
For 9,000 linear feet tramway	$31,800	
Plant charge per foot		3.53
		$17.03

In addition to the cost of the tramway over the actual length of the jetty to be rebuilt, about 9,000 feet, it would have been necessary to raise the shore tracks across the sand on a shore trestle, amounting in all to about 7,000 linear feet at an estimated cost of $10 per foot. While the cost of the shore track at elevation 12 has been—

For labor	$1.13
For rails, spikes, and ties	2.50
	$3.63

The low elevation for the shore tracks possible with the system used has therefore effected a total saving of $44,590 over the cost of the shore trestle necessary to reach a tramway 24 to 28 feet above low water at the ocean beach line.

The desirability of making full use of the existing enrockment with its established slopes and compact mass, as well as the advantage of having a thoroughly stable foundation for the stone unloading crane, and the high cost of the tramway of sufficient strength to permit the use of the large stone proposed, were the conditions which led to the adoption of the concrete cap.

The main idea of the concrete is to hold the track when the jetty is swept by waves. The only portions of the structure that will float are the ties, and these are firmly imbedded in concrete

and held in place. A further advantage of the concrete cap method is that the ties are the only portion of the structure which is not of a permanent nature and an addition to the value of the structure as a jetty. The concrete top on the crest of the structure will greatly retard the unraveling action of the waves, and when finally broken up by the settlement and washing away of the rock slopes it will still be of value as jetty material.

The cost of the concrete cap and tracks has been:

For labor		$2.86	
4½ tons Class 3 stone in voids of jetty enrockment	$6.75		
2 tons of Class 4 stone	3.00		
.95 barrel cement	2.00		
Total concrete material		$11.75	
50 feet B. M. lumber, at $14 per M.	.70		
Fuel, oil, repairs, etc.	.66		
		1.36	
			$15.97
Rails and joints (recoverable)			2.13
Total cost per linear foot			$18.10

The above estimate includes the cost of portions of track lost by wave wash before the concrete had time to set, amounting to 1 per cent.

The plant cost for the concrete method had been:

One concrete mixer, complete with boiler and engine	$1,500
One flat car	600
Tanks and small tools	250
Labor assembling plant	200
Total	$2,550

Cost per foot for one jetty, 53 cents.

The concrete method of construction has proved to be all that was expected of it. The length of time required for its construction has been less than was estimated, although the cost has been more per foot. The advanced cost, over the estimated cost, is due to the use of a greater quantity of Class 3 stone than was estimated. Experience has shown that the use of Class 3 stone for the top course is more economical than the placing of Class 2 stone, so that it will not interfere with building track.

The 2,400 feet of jetty with concrete cap already built has been subjected to very severe weather since September 1. Waves that displaced stones weighing as much as 5 tons from the slopes and

moved them to the top of the track have repeatedly attacked the jetty. So far, there has not been the least sign of deterioration except at places where the concrete, owing to the immediate use of the tracks while it was still plastic, has parted at the bottom of the ties and the corners have been chipped off. These slight defects in no wise damage the structure and are not of a nature serious

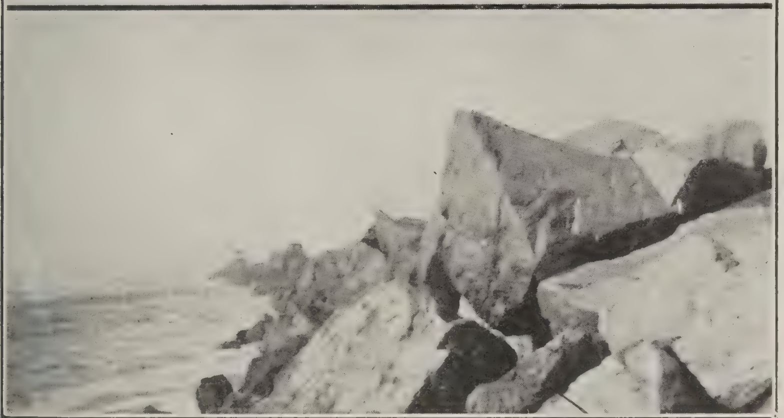


Fig. 5 (upper). Track arrangement on jetty receiving apron and approach.
Fig. 6 (lower). Sea slope of jetty.

enough to call for repairs, now or in the future. At several places the stone on the slopes has been washed away, leaving a vertical side at the line of the cap, or it has been undermined 2 or 3 feet. These places have been repaired with stone, hand-placed where necessary, without breaking the concrete.

Some loss of freshly laid concrete has occurred when it has been

exposed to severe wave action within four or five hours after placing. This loss has not been sufficiently great to add materially to the cost. One severe storm carried away 25 feet of the outer end of the track, but to all appearance the damage was caused by unraveling the enrockment which allowed the concrete to break down.

The method of construction is: The enrockment is first brought to an elevation averaging 2 feet below the finished grade with Class 2 stone; pieces weighing from 1,000 pounds to 10 tons. None of the pieces of Class 2 stone are allowed to project above 6 inches below grade—the bottom of the ties. Voids in the mass are then filled with Class 3 stone, pieces weighing from 3 pounds to 500 pounds, and the top is leveled off at from 18 to 10 inches below grade. Holes are choked by hand-placed stone. A rough form is made by tying wale pieces, 6 by 8 inch by 20 foot ties, together with wires 5 feet apart, and nailing to them short vertical boards with bottoms in contact with the rock. A rock dam is built at the front end of the form. Concrete is mixed rather dry, deposited with a cubic yard self-dumping and self-righting bucket, handled by the stone-unloading crane. The concrete is brought to within an inch or so of the bottom of the ties. The end tie is brought to grade, the crane rails laid, and the ties placed and spiked. Concrete is then continued to the top of the ties.

The concrete mixer is mounted on the end of a standard flat car, the discharge shoot delivering over the end. Mixing water is supplied by gravity from a tank on the opposite end of the car. Oil fuel is supplied by gravity from a tank near the water tank. The oil and water tank also supply the stone unloading crane. Oil is pumped and water flows by gravity to the crane supply tanks. Cement for a day's operation is carried on the concrete mixer car.

The concrete is machine mixed in a Foote Batch Mixer of 21 cubic feet capacity, end-discharge type, steam driven.

At the outer end of the jetty there are two working tracks. All material and equipment are regulation master-car builders' pattern and dimensions. The tracks are 14 feet 7¾ inches, center to center. When in operation the concrete mixer car occupies the left-hand track, and the car containing the aggregate the right-hand track. A working movable platform fills the space between the cars, leaving about 1 inch clearance over the stake pockets. The elevation of the platform is the same as that of the car decks. The aggregate is shoveled directly from the cars into the charging

Fig. 7. Concrete aggregate and track material.

skip and a large barrow. The charging skip is marked by a row of rivets at the height containing the charge required. In order to work a sufficient number of shovelers to keep the depositing bucket in motion it was found necessary to provide a greater length than was possible by shoveling into the skip alone. A two-wheel barrow, running on rails and holding about 15 cubic feet, was mounted on the mixer car. This allows the charging shovelers to be distributed over the whole length of a standard flat car.

The most economical crew for depositing concrete is eight laborers charging aggregate, one laborer charging cement and one engineman operating mixer. The concrete is placed, spread, and tamped by the regular stone unloading crew, consisting of engineman, four laborers and the foreman. This crew will generally build a section of track 18 to 20 feet in length in two hours and fifteen minutes, including the construction of forms and placing ties and rails.

Under the specifications for the material the contractors are allowed to supply either broken stone, crusher run, or unscreened river gravel for aggregate. The material supplied is tested for proportioning in the following manner: The aggregate as delivered is screened and the portion passing a plate containing ¼-inch diameter holes is considered sand, and the balance stone. If necessary, sand is added to form a mixture corresponding to a 1:2½:5½. The gravel supplied contains rather larger proportion of sand than is required. When broken stone is delivered it is necessary to add about 16 pounds of sand per 100 pounds of aggregate as received.

Tests of the material delivered are made from time to time to form a reasonably close record of the proportions of the aggregate. This very rough system of measuring the material has resulted in a very fair concrete, the requirements of the work being considered. A more refined method of proportioning the materials, while perhaps resulting in a denser concrete, would render necessary screening and measuring plants and labor for handling and it is doubtful if any real economy would be effected. In order to accelerate setting during severe weather when the fresh concrete is exposed to wave wash within a few hours after depositing, the cement portion is sometimes increased.

Concrete displacers, stone of from 3 pounds to 200 pounds, are added to the mass below and between the ties.

Fig. 8. Concreting extension of track; crest of old jetty at low water in background.

Stone for the work is supplied under contract by the Hammon Construction Company at the following prices:

	Per ton.
Class 1	$1.74
Class 2	1.56
Class 3	1.50
Class 4	1.50

The following is printed from the specifications under which the stone is being supplied:

DESCRIPTIONS OF MATERIAL.

Class 1 Stone. To be of large pieces only, weighing from 10 tons to 20 tons each piece. These stones will be used for slopes on the outer end of the work, and none will be received until the jetty repairs have been extended 2,300 feet from the high water shore lines. Delivery of this stone will be required up to 500 tons per day when work is in progress on the outer ends of both jetties. Stone of Class 1 will be loaded directly on the flat car without the use of skips.

Class 2 Stone. To be in pieces of 1,000 pounds each to pieces of 10 tons each, in the following proportions:

One-fourth of each day's supply may be in pieces of 1,000 pounds to pieces of 3 tons; one-half of each day's supply must be in pieces of from 3 tons to 6 tons each; and one-fourth of each day's supply must be in pieces from 6 to 10 tons each.

This class of stone will form the major portions of the repairs to the jetties. It is expected that the delivery required will not be less than 1,000 tons per day after the work has been well commenced on both jetties, and about 500 tons per day for the first season's work.

Stones of Class 2, when in pieces under 6 tons each, must be loaded on suitable skips holding up to 10 tons of stone each. Skips will be strong and designed for lifting at the four corners. Four corner hooks will be provided on each skip for the convenient attachment of the unloading crane spider chain. Skips will be kept in good working order by the contractor.

Class 3 Stone. Will be used only for bringing the top of the rough mound to a nearly smooth tight surface and for concrete displacers. It will be in pieces not less than 3 pounds nor more than 500 pounds each, delivered on skips similar to those above described. Stone of Class 1 or Class 2 must not be loaded on cars carrying stone of Class 3. Stone of Class 3 will be used in decreasing amount as the jetties are extended. From 120 tons per day at the commencement of the work to about 10 tons per day when the outer end of the work is reached.

Class 4 Stone. Will be used for making concrete. It may be clean crusher run of the same rock as used for the other classes, or

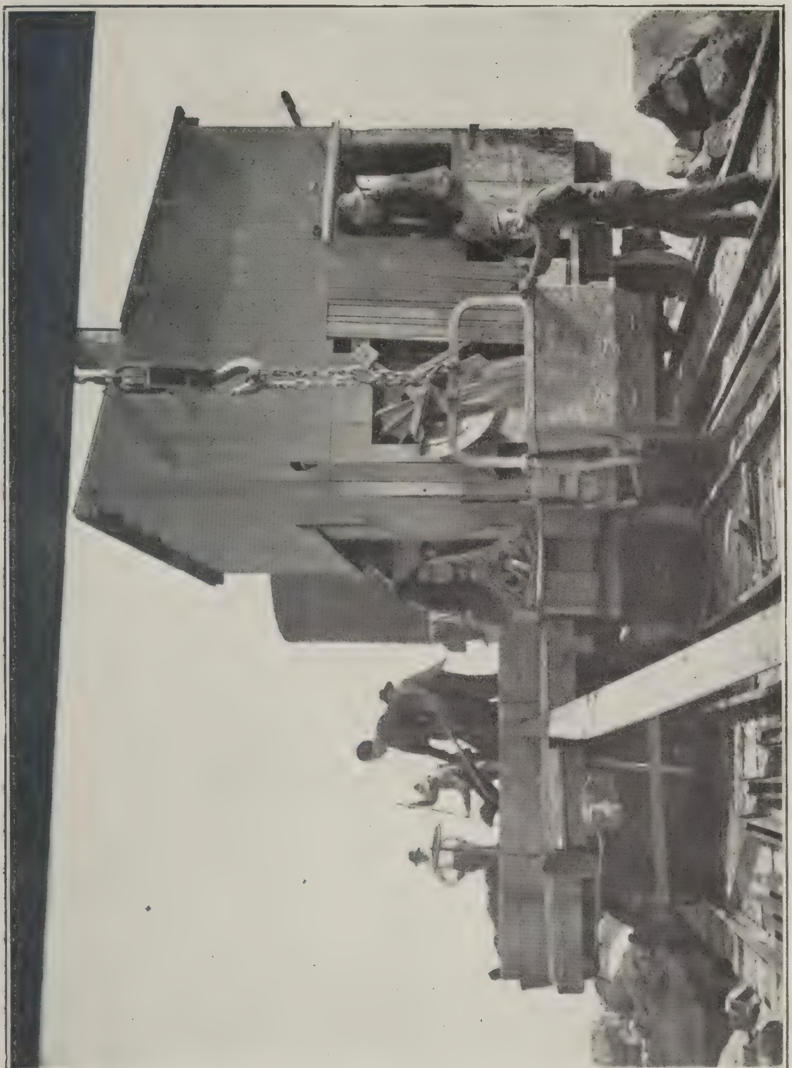


Fig. 9. Concrete mixing plant discharging.

a good selected or washed river gravel may be used. It must vary in size from pieces with greatest dimension not more than 3 inches to the finest product of the crushers. If river gravel be used it must be thoroughly washed, all organic matter of any nature removed, and screened if necessary to exclude pieces greater than 3 inches largest dimension. Either gravel or stone must be of hard durable rock which will not disintegrate in the finished work.

Broken stone or gravel will be loaded on flat cars, provided with suitable sides. The amount required will vary from about 70 tons per day during the first part of the work to 8 tons per day for the outer ends of the jetties.

Stone of all classes must be hard, close grained, and not liable to disintegrate in the work. It must be free from seams of any nature and of uniform grain and texture. It must be delivered free from all dirt, quarry refuse, or foreign material of any nature, preferably in nearly cubical blocks. The greatest dimension of any piece will not exceed five times the least dimension. It must weigh not less than 150 pounds per cubic foot, dry. Bidders will submit samples of the stone they propose to deliver and will name the location of the quarry or quarries which they propose to operate. Stone will not be accepted for use in the jetty until the quarry from which it is to be supplied has been approved by the contracting officer. A compact stone of high specific gravity is desired. The weight per cubic foot and quality of the rock, as well as the price per ton, will be considered in making the award.

The United States reserves the right after at least a season's trial to require that no further deliveries of stone of Classes 3 and 4 be made, but that the amount of stone to make up the undelivered portions of the 30,000 tons and 14,000 tons mentioned in paragraph 18 be furnished of Class 1 or Class 2, or partly of each class.

The approval of a quarry by the contracting officer shall not prevent the rejection of any stone not complying with the requirements herein specified.

The larger portion of the stone supplied is a close-grained igneous rock, weighing about 198 pounds per cubic foot. It is very difficult to quarry, breaking into very uneven fragments. However, by proper manipulation, a minimum of waste is secured and, as there is no covering soil to be contended with, the quarry is very satisfactory.

A second stone supply from the same vicinity is a close-grained metamorphic sedimentary rock with irregular planes of division of argillaceous material. This stone weighs 167 pounds per cubic foot and is easily worked. On account of the seams and a considerable covering of soil, which cause a large amount of waste, it

has not been found advantageous to furnish any considerable quantity of this stone so far.

The stone is loaded by the contractors on standard flat cars and hauled 7 miles to a loading point on a navigable channel, where it is placed on barges carrying eight cars each. When delivered at

Fig. 10 (upper). Geared locomotive, Contractors' Plant, lowers three loaded cars down 8 per cent incline.

Fig. 11 (lower). Contractors loading derrick at quarry.

the jetty receiving plant, it is unloaded and the empty cars are returned to the barges. From the loading point to the jetty landing is 9 miles.

The above arrangement permitted the contract to be made for

the material only and the contractors have nothing to do with the actual jetty construction. The United States is not required to pass on the rock until it is offered for use at the jetty wharf.

The contractor's crew has numbered generally about one hundred men, employed for seven ten-hour days per week. The following plant has been installed by the contractors:

Four 20-ton stiff-leg derricks, 100-foot booms, with steam-driven hoisting and swinging engines.

One steam crane, with shovel attachment, used for grading pits and tracks and for loading cars.

One two-stage, 16 by 10 by 14 inch, Ingersoll-Rand cross compound air compressor, electrically driven.

One small jaw rock-crushing plant, electrically driven.

In addition, there is the usual equipment of air drills, small tools, shop and mess equipment and appliances.

Hollow drill bits are used, and since the installment of an air-driven Leyner sharpener no difficulty has been experienced in successfully quarrying the stone. The contractor's transportation plant consists of fifty flat cars, 60,000 pounds capacity, 36 feet long, two car ferry barges and two tow boats. From the quarry to the landing the cars are handled over a logging railroad by the logging companies' motive power.

The contractors are now providing fifteen additional cars and a third barge.

The receiving and depositing plant at the jetty, belonging to the United States, consists of a 100-foot span, three track apron for transfer from barge to shore tracks, adjusted to tidal elevation at barge end by counterweights, and fixed at 10-foot elevation at shore end; two locomotives; three flat cars for miscellaneous materials; a 20 cubic-foot concrete mixer mounted on a flat car; a 10-ton revolving and traveling unloading crane, gauge of gantry 14 feet; water supply and distributing system; fuel oil storage and distributing system; a repair plant with power driven tools for ordinary blacksmith, carpenter, and light machine work; an electric light plant; store house, mess house; crew quarters and necessary minor equipment for the work in progress. A new stone unloading crane of 20 tons capacity is now in course of construction.

The jetty crew varies from forty to fifty men working six eight-hour shifts per week.

To January 31, 1913, the contractors have delivered the following

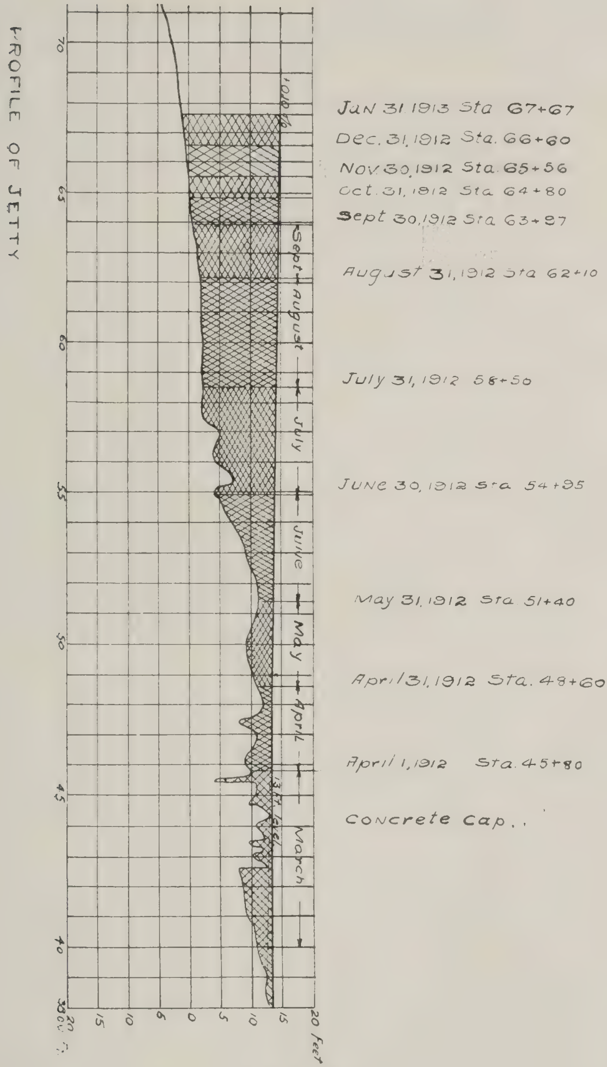

PROFILE OF JETTY
Jan 31 1913 Sta 67+67
Dec. 31, 1912 Sta. 66+60
Nov. 30, 1912 Sta. 65+56
Oct. 31, 1912 Sta 64+80
Sept 30, 1912 Sta 63+27
August 31, 1912 Sta 62+10
July 31, 1912 58+50
June 30, 1912 Sta 54+95
May 31, 1912 Sta 51+40
April 31, 1912 Sta. 48+60
April 1, 1912 Sta. 45+80
Concrete Cap.
Sept
August
July
June
May
April
March
20 Feet

Fig. 12.

quantities of stone which have been deposited on the south jetty:

	Tons.
Class 2	87,545
Class 3	15,024
Class 4	4,460

Fig. 12 (p. 517) is a profile of the South Jetty showing the prog-

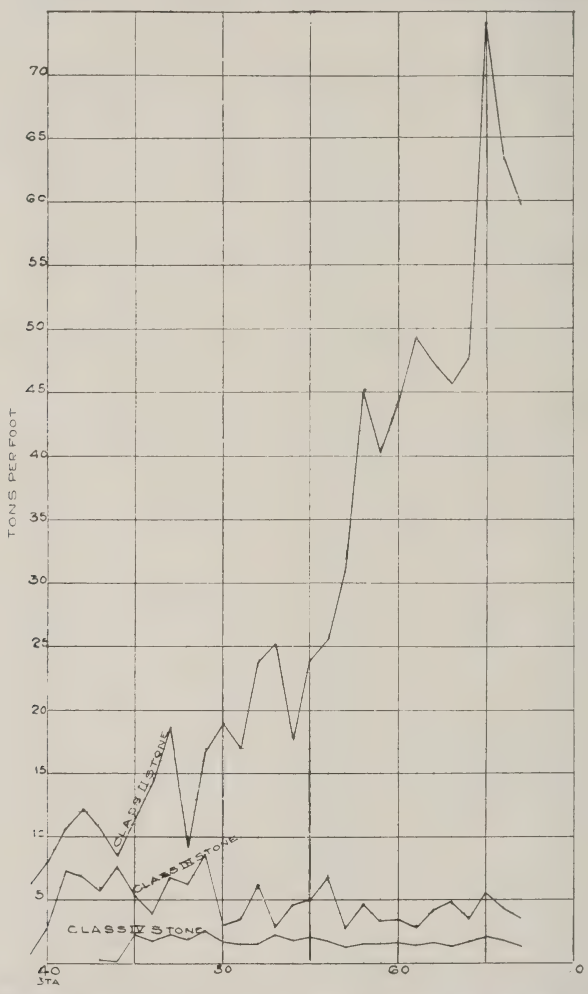


Fig. 13. Showing amounts of different kinds of stone put in per foot of jetty, zero being at shore end.

ress of the work, and Fig. 13 (page 518) shows graphically the amount of stone of different classes used per foot.

This work has been under the direction of the following officers: Col. John Biddle, Lieut. Col. Thos. H. Rees, and First Lieut. Thos. H. Emerson, Corps of Engineers, U. S. Army.

Notes on Field Fortification

BY

Maj. W. W. HARTS
Corps of Engineers; Member American Society Civil Engineers

GENERAL PRINCIPLES.

There can be no doubt that the broad purpose of fortification is thoroughly comprehended by most army officers and that we very properly recognize in it a means of artificially strengthening a defensive position, in order to economize troops so that strong reserves will be available for the decisive action.

The various kinds of trenches probably also will be assigned their correct tactical application,such as the hasty trench to assist in holding a temporarily seized line in an offensive movement, the firing trench to hold more tenaciously a previously prepared site, and the fortified pivot or supporting point to resist distant artillery fire while sheltering the infantry troops which are awaiting the hostile infantry assault. But on the other hand, when called on to put our ideas into effect practically, a cloud of difficulties often arise in the minds of most of us.

It is thought that a review of some of the elementary principles will be of service in clearing away a portion of these difficulties.

Let me start by saying, first, that all field fortification is only an accessory to the tactical requirements of the situation presented; is only a means to an end and not an end in itself; a servant, not a master; a handy tool and not a shackle; and must always be so regarded.

When well understood, its value to the commander is always great, and its aid is often indispensable, but it should be remembered that fortification exists for tactics, not tactics for fortification.

Secondly, it may be added, all works should be simple. Not only is construction easier for the troops, but in all field fortification time is an element of enormous value, and simplicity saves both labor and time. It is, of course, well understood that simplicity in

all engineering work is the essence of good design, and it scarcely needs to be said that the complex and ill-understood details in field fortification are invariably to be avoided in favor of the simple and the plain. The technics of the art should never overshadow its purpose.

Thirdly, it is unwise to counsel perfection. It is not so bad to locate a trench wrong in some particulars if it can still be used effectively, but what sort of engineering would it be called should we take so much time in trying to select the best possible site or the best theoretical cross-section, that we will be unable to finish the trench in time to use it advantageously? Prompt decision in the selection of the site and energy in execution are of prime value. Experience is lacking with many of us in the actual use of trenches in war and all ideas are, therefore, based mainly on the experience of others, as this is a practical subject. But it is far better to use one's intelligence in formulating sound ideas on fortification principles and in investigating for one's self the reasons for the various types and uses of trenches, than to vainly attempt to remember some rigid or complex rules which may so hamper initiative as to defeat their purpose.

Fourthly, troops under fire can not remain stationary without some kind of cover, either artificial or natural. On this principle rests the justification of all the labor and expense that are so often necessary.

TACTICAL USES OF TRENCHES.

Field works to-day may approach near to semi-permanency. As line officers, we are more particularly interested in trenches which are constructed by troops in the field with the means at hand. Among these works are the hasty and the firing trenches, with perhaps some of those improvised structures which provide stronger and more reliable cover for special points. Permanent works will be rarer in the experience of line officers, as they tend more to the nature of protection of strategic points when time is usually ample and more deliberate work is possible. They usually fall within the province of the engineer, although it should, of course, be remembered that there is no hard and fast rule for dividing trenches into definite classes as has sometimes been done. They all merge into one another by imperceptible degrees or develop from one into another in use. In fact, it seems plain that ordinary field trenches may with time be so strengthened as to furnish all that is usually

demanded of permanent land defense. The skirmisher's pit or the rough stone pile, log or mud wall afford examples of one extreme of which the elaborate seacoast fort is the other.

There are two aspects of all fortification work, viz: the tactical and the technical. Let us first examine the tactical.

TRENCHES ON THE OFFENSIVE.

Although trenches are thought to be peculiarly a defensive measure, they are not exclusively so, as they are often used in offensive movements with excellent effect.

The hasty or skirmisher's trench is often built under fire, and almost never built except under extreme urgency. It may at times, through necessity, be located without much attention to the ground and without much care as to its shape or profile, but even then a good general line should be maintained and a good field of fire secured notwithstanding the fact that cover and concealment must be obtained quickly.

The Japanese were often seen to pause in their rushes during the attacks on the Russian fortified positions, lie down long enough to dig a pit, pile up the earth before them and thus obtain some cover. In advancing under fire the Japanese troops rush forward in skirmish lines, drop to the ground along a line selected and held by the non-commissioned officers, and each alternate man fires at the enemy, while the other digs a pit in a lying position. These pits are connected by means of a trench and improved by succeeding waves of skirmishers.

Usually in the later stages of this war advances over the open ground were too costly and entrenchments were made in new positions after dark, for which daylight reconnaissances had been conducted beforehand.

These skirmishers' trenches seem to be the most closely identified with tactics of any, and give some justification for calling the spade an additional arm of the service. Digging these hasty trenches is receiving much encouragement among foreign armies, for it is being realized that such trenches will be more and more necessary in the attack in the future, if infantry fire continues to improve in volume and accuracy as it has in the past.

As far back as the Atlanta Campaign in the Civil War, the troops on both sides made effective use of hasty trenches on the offensive. No sooner had they gained a position than it was at once entrenched. Their three years' experience had taught them where, how, and

when to entrench. Sherman, in his Memoirs, says: "Troops halting for the night or for battle, faced the enemy, moved forward to ground with a good outlook to the front, stacked arms, gathered logs, stumps, fence rails, anything which would stop a bullet, piled these to the front and digging a ditch behind, threw the dirt forward and made a parapet which covered their person as perfectly as a granite wall." While holding the Confederates in their trenches before Atlanta by a vigorous rifle and artillery fire, Sherman's trenches were frequently extended beyond one or the other of Johnston's flanks, and the Confederates forced out of their lines, often without an assault.

In the Boer War, the capture of Gen. Cronje, at Paardeburg, was effected by the systematic use of trenches on the offensive after assaults in the open had failed. After surrounding the Boer position with an entrenched line some days before, a small force of English were enabled one night to secure by a daring attack a position in close proximity to the Boer lines. This they immediately entrenched, although under heavy fire, and by daylight enough men were in a secure and advantageous position to enable them to enfilade the Boer trenches with such effect that the Boers were forced to surrender before the next night.

In Manchuria, the Japanese entrenched in one night seventy-two field guns and twenty howitzers on an island in the Yalu River directly in the Russian front, and at daylight smothered the Russian fire in a very unequal combat, during which the Japanese batteries were not hit once nor even found, they were so well concealed.

The adoption in most armies of the modern, quick-firing, short-recoil field gun has made battles progress more slowly and last longer in recent years than formerly. They have forced the infantry line to seek cover at a greater distance. In the advance on an entrenched position, it will be difficult for the attacking force to tell positively from what quarter the hostile fire comes. Smokeless powder, concealed trenches, and artillery firing by indirect laying will so conceal the defenders that but little will be seen, and the noise and hits of the shrapnel shells and rifle bullets may be the only tangible evidence of the defenders. For this reason some concealment of the attacking force is likewise necessary. Whenever a forward movement is undertaken against a well-defended line, it must invariably be from point to point, each step to be firmly secured and covered before the next step is attempted. For this

purpose trenches of some kind will be found indispensable in open country.

In the English Manual of Combined Training the tactical plan of attack recommended is, "To outflank at least one of the enemy's wings, to seize localities from which a searching and sustained fire may be developed against a weak point of his position, to strike at that point heavily, unexpectedly with the greatest strength possible, and elsewhere to establish bodies of infantry so close to the enemy's line as to hold him to his ground and to prevent him from either changing front or from reinforcing the troops defending the line of attack." In every stage of such operations the various separate steps can be secured and held only by using cover. If the points are seized in the daytime some kind of a skirmisher's trench will be found necessary to hold the position, until night arrives, when concealment will be afforded for strengthening the position and for constructing communicating trenches, by means of which reinforcements may be brought in by day. If seized at night deep standing trenches will likely be constructed at once, to be held tenaciously the next day. In any case, such an advance can only be made by using some kind of cover, of which the ordinary standing trench will probably be relied on oftenest. If the country is level, more digging will of course be necessary, especially if the supports and reserves have to be protected by trenches or parapets, than where folds and undulations in the ground provide some natural cover.

DEFENSIVE USE.

We have seen that trenches may be used effectively on the attack, but it is on the defense that their use is especially applicable. Furthermore, on the defense their use is usually better and more widely understood. A warning has been given, however, by some military writers that trenches tie the troops to the ground too much, render them less mobile and more averse to leaving their positions for an offensive movement. This, of course, applies with more force to untrained troops than to seasoned soldiers, and the remedy is certainly not to abandon altogether the help that trenches afford. In this connection it should be borne in mind that decisive results can seldom be obtained by defensive action alone and fortifications must never be allowed to dominate tactics. The possibility of assuming the offensive should never be abandoned by the defenders. To adopt a phrase from an English text-book,

"The hornets' nest rather than the hedgehog should be the defenders' symbol. It is not enough to present a bristling front to the enemy, he must be stung if he approaches too close." In the siege of Mafeking during the Boer War seven hundred men within a line 5 or 6 miles long around the city held back a Boer army of eight thousand men and ten guns for a month, and held out for six months longer against four thousand men and six guns, inflicting a loss reported at over one thousand killed and wounded. During this siege one of the Boer attacks was met by a counter-attack in which one hundred and eight Boers were captured.

It would manifestly be a mistake for an alert commander not to inflict a loss on the attackers if a favorable opportunity should present itself, and in fact one of the purposes of entrenchments is to enable an effective reserve to be held for such contingencies.

On the other hand, it was plain from the operations in Manchuria that strongly fortified positions can not be taken by frontal attacks, except after long delay and much loss. They will yield only to a long artillery bombardment and a step-by-step advance of the infantry. This usually makes a flanking movement necessary, and time and again the Russians abandoned carefully prepared positions as soon as the Japanese flanking movement became effective.

DIVISION INTO SECTORS.

The tactical arrangement of defensive positions deserves close attention. Every defensive line, whether bent around some position it is protecting or extended along the front of an army should, as a rule, be divided into independent sections or sectors of such size as can be well commanded by one man. In this way tactical units with their supports, reserves, supply, etc., may be kept together and the duty of attending to the multitude of details may easily be divided by the commander among a number of subordinate officers. This separation of the defensive front is needed mainly for better field control, but it should extend to administration and supply as well.

The outer line of trenches would ordinarily be manned in groups, thus enabling units to be kept intact. For example, out of each battalion assigned to the trenches we may expect either one or two companies to form the supports, depending on the nearness of the enemy, the natural strength of the position and the likelihood of attack. These supports are kept in a concealed place behind the line of trenches, within about 300 yards, in order to be within easy

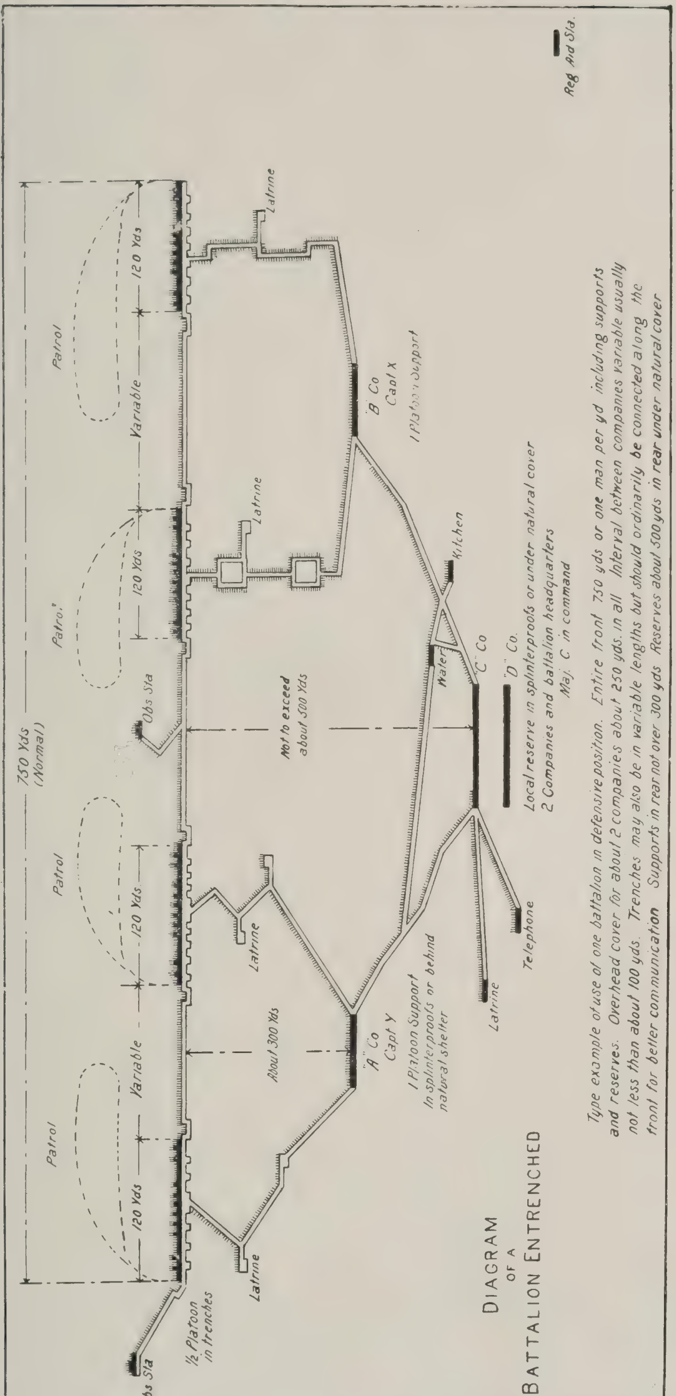


DIAGRAM
OF A
BATTALION ENTRENCHED
750 Yds
(Normal)
Patrol
Patrol
Patrol
Patrol
120 Yds
Variable
120 Yds
120 Yds
Variable
120 Yds
Obs Sta
Obs Sta
½ Platoon
in trenches
Latrine
Latrine
Latrine
Latrine
Latrine
Not to exceed
about 500 Yds
About 300 Yds
"A" Co
Capt Y
1 Platoon Support
In splinterproofs or behind
natural shelter
"B" Co
Capt X
1 Platoon Support
Kitchen
Water
"C" Co
"D" Co.
Local reserve in splinterproofs or under natural cover
2 Companies and battalion headquarters
Maj. C in command
Telephone
Reg. Aid Sta.
Type example of use of one battalion in defensive position. Entire front 750 yds or one man per yd including supports
and reserves. Overhead cover for about 2 companies about 250 yds. in all. Interval between companies variable usually
not less than about 100 yds. Trenches may also be in variable lengths but should ordinarily be connected along the
front for better communication. Supports in rear not over 300 yds. Reserves about 500 yds in rear under natural cover

Fig. 1.

reach in case of an attack. Periodically they change places with the men in the trenches, to relieve them of the strain of being constantly under fire. Supports will be protected usually behind a fold of the ground and have protected communicating trenches with the front line. Out of each regiment assigned to a section we may expect about one battalion to be held as a section reserve, or if a larger force is assigned to a section about one-third or one-half may be selected for a section reserve. This reserve should be at the headquarters of the section, and be near the center, where it is easiest to reach the various units on the front line. If the position is to be occupied for some time, it should have wire communication with the supports of the larger separate units in the trenches, such as battalion supports, and should be connected with the headquarters of the command and with the various supply depots.

There seems great need of supplying each regiment with a simple telephone outfit to enable it to reach at least three points from the regimental headquarters. Although this has been proposed before, no steps have yet been taken in our army toward enabling a regimental commander to communicate with his principal units, which are often scattered over a long stretch of front. If the time allows and the site favors it, a road a short distance in rear of the first line of supports should be constructed for better communication along the front.

GENERAL RESERVE.

A strong general reserve consisting of complete units, as far as practicable selected in turn from the entire command and amounting to as much as one-half of the entire force, is strongly advocated by many authorities. In case of a persistent attack on a weak place in the line the local reserves and general reserves might be brought into the front line of trenches and would be likely to be held there during the attack. If, then, the commander has no other reserve force left, he has lost all hope of influencing the combat by a counter-stroke and may not be able to meet any new developments on the part of his adversary.

It is considered better to strengthen a hard-pressed part of the line by moving troops from another part not so strongly attacked, rather than rush in the general reserves too soon. The necessary weakness of a long line inadequately defended, which will be necessary at times in modern warfare, will frequently be largely neutralized by a strong mobile reserve moving on interior lines

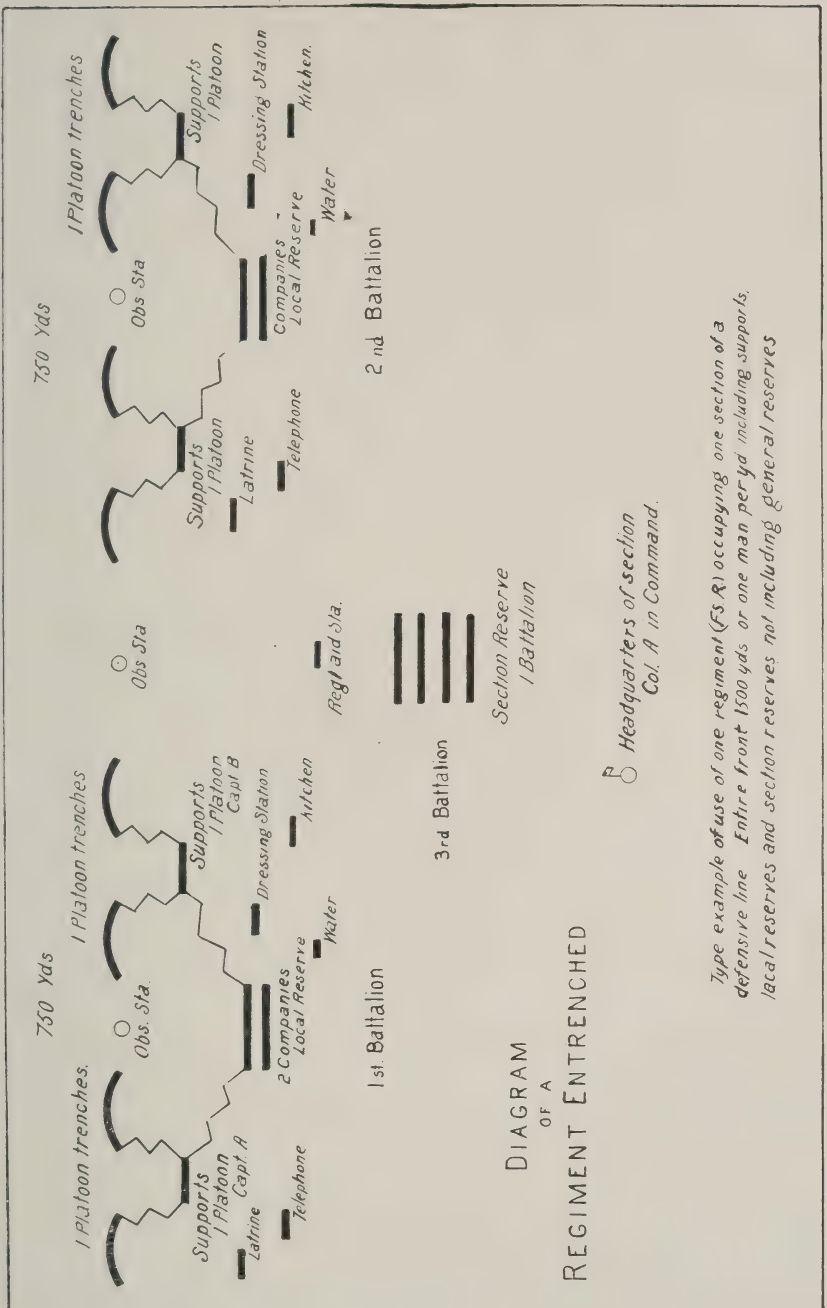


DIAGRAM
OF A
REGIMENT ENTRENCHED
750 Yds
1 Platoon trenches.
1 Platoon trenches
Obs. Sta.
Supports
1 Platoon
Capt. A
Latrine
Telephone
2 Companies
Local Reserve
Water
Dressing Station
Kitchen
Supports
1 Platoon
Capt B
1st. Battalion
Obs Sta
Reg't aid Sta.
3rd Battalion
Section Reserve
1 Battalion
750 Yds
1 Platoon trenches
Obs Sta
Supports
1 Platoon
Latrine
Telephone
Companies -
Local Reserve
Water
Dressing Station
Kitchen.
Supports
1 Platoon
2nd Battalion
Headquarters of section
Col. A in Command.
Type example of use of one regiment (F.S.R.) occupying one section of a
defensive line. Entire front 1500 yds or one man per yd including supports,
local reserves and section reserves not including general reserves

Fig. 2.

along good communicating roads, ready to reinforce vigorously any threatened point.

To summarize, therefore, from one-quarter to one-half of the entire force should constitute the general reserve and the remainder be distributed in the various sections of the front as needed. In each section it is advocated that about one-quarter to one-half of the force assigned to it should constitute the section reserve, and of the troops remaining about one-half or two-thirds should be supports. About one-fourth to one-sixth only of the section's force will actually be in the trenches before being attacked. It seems to be desirable to reduce the force of men actually in the trenches to the smallest number consistent with safety. No exact proportion of reserves and supports can be given for all cases and the foregoing is only stated as a guide in forming one's own decision, which must always depend on the conditions peculiar to the ground, including the shape of the front, whether straight or curved, the size of the garrison, the enterprise of the enemy and the interior communications.

But room in the trenches must usually be made for all the troops in the section, including perhaps space for some of the general reserve, so there must be some calculation made as to how much trench must be built or how much front can be occupied.

Trenches must always be able to furnish enough aimed rifle fire to stop any ordinary infantry attack across the open space in front. It is plain that this will vary greatly in different terrain. One might suppose that the enormously increased power and rate of fire of the modern rifles had lessened the number of rifles necessary, and that the old rule of thumb of one man per pace of firing line is no longer required. Such density is no longer necessary, except perhaps for a short time for repelling assaults after the attacking troops have managed to bring a large force within close assaultng distance. At such times the maximum amount of fire possible is needed. Men can not use their rifles accurately if occupying less space than one pace of the front. This may then be adopted as a maximum density required for any part of the front where a close attack may be delivered. This assumes that a part of the general reserve is on the line. At other places less hard pressed more length of trench and less density will be possible. An English authority has even stated that one man to every eight or ten paces is enough to check any infantry assault on a well chosen position, or including reserves and supports from two to five paces per man.

We may safely conclude that from 1 to 4 yards per man, including sectional reserves and supports, are limiting estimates depending largely on the ground. The density suggested in our field service regulations, including supports and local reserves, is one man per yard, giving a front of 750 yards for the battalion, including intervals of 100 yards between companies, 1,500 for the regiment, and 4,500 for the brigade. The judgment of the officer as to the strength required will have to be exercised in each of the larger parts of the line in order to have it fairly balanced, increasing the density in some places and decreasing it in others.

If we assume that the brigade is to be the defensive unit assigned to a section of the line, it would cover a front varying from about 2½ miles to perhaps four times that length, depending upon the size and mobility of the general reserves, the natural strength of the position and the aggressiveness of the enemy.

LOCATION OF TRENCHES.

Having examined briefly the tactical application of entrenchments to the offensive and to the organization and strength suitable for the defensive, let us take up for a moment the most desirable location for trenches or "siting," as it is called by some. The first and most important desideratum is a good field of fire. Without this no trench can be called well placed. It is the primary purpose for which the trench is constructed and should have the first consideration. The field of fire is not a good one unless it permits the full use of the weapons of the defenders over a zone wide enough to prevent a successful assault. This requires a level or slightly sloping open space in front for 400 to 600 yards. Fire over a parapet is not very accurate when the rifle is held at a considerable angle with the lines of the work, such as 15° or over. Parapets should therefore be fairly straight and at right angles to the fire. There should be no dead space in front which is not covered by a neighboring trench. For best effect fire should be grazing or parallel to the ground and not plunging from a height.

The second requirement in good location is concealment. This is favored by low parapets with some sort of background, narrow trenches, avoidance of sky lines, absence of sharp crests and angles, low site and a treatment of the disturbed soil and parapets to make them correspond in appearance with the surrounding ground. Time and labor spent in concealment are always well rewarded.

The third requirement is safe cover. This is obtained by making

the parapets thick enough to resist penetration, by making deep and narrow trenches with steep sides, by building many comparatively short lengths of trenches separated from each other by an intervening stretch of undisturbed soil as a traverse, and by widening the trenches under the surface as practiced by the Boers, and if time permits it by providing overhead cover.

It requires about 3 feet of earth to stop a rifle bullet; about double that, 6 feet, to stop a shell of a light field piece, and about double that again, or 12 to 14 feet, to protect against heavy artillery fire. All these distances are, of course, measured along the line of fall. About 6 or 8 inches of compact earth will protect against shrapnel bullets and splinters. A slope of about 1 vertical to 5 horizontal or flatter in the pathway of an artillery projectile will almost invariably deflect it upward, and if the parapet is so designed as to present such a slope, especially if inconspicuous, thinner parapets may often be adopted with reasonable safety.

A fourth requirement is to locate the trenches with a view to having them reinforced or emptied unseen from the front. High sites favor this requirement and are usually to be selected for rear guard actions, although low sites are always to be preferred for more aggressive defense.

Where the ground is undulating a location for supports in rear can usually be found where they may be screened from view and covered from fire by the shape of the ground. It will often be found that communicating trenches will be needed to reach these protected areas from the firing trenches, and these communicating trenches will usually have to be broken and sometimes zigzagged or traversed to avoid enfilading fire. Cover trenches for the supports and sometimes for the reserves will be needed in flat country, as some sort of concealment for reinforcements is absolutely necessary.

A fifth requirement should be to select the site so as to force the enemy to advance over a narrow and well swept front, over difficult ground, streams or cleared land, where he will be exposed as much as possible, where his fire can not be well developed and where his free movement will be impeded.

It is plain that all these requirements seldom can be met, but it seems equally plain that the location meeting most of them will be the best. Selection of sites may always be expected to involve a balancing of advantages against disadvantages. To obtain the same view of the field of fire that the man in the trenches will have

it is necessary to select locations by placing his eye on his level. The outlook from a standing position on the surface is sometimes very different from what it is in the trenches and may even require the trenches to be moved forward or backward to uncover the dead space often found behind intervening hillocks. Preliminary locations should always be tested by examining the ground from a sighting position about a foot or so from the surface.

In concealing trenches, too, it is always desirable if it can be done to go to the front and view the trenches from nearby hills at about artillery range. Even if no more than a few hundred yards are possible, it will often indicate what, if anything, is still desirable for more complete concealment.

HIGH OR LOW SITES.

There has been some discussion from time to time as to which is preferable, a high or a low site for trenches, and the best test to apply is which gives the best field of fire. Grazing fire, nearly parallel to the surface, is generally believed to be most effective for small arms, as it increases the danger space. This leads to the usual selections of low sites in spite of the fact that greater pains must be taken in constructing connecting trenches, that more difficulties will be met with in reinforcing or emptying the trenches, and that they may be more easily searched with hostile shrapnel. But on the other hand they are more easily concealed, having a better background and there is usually more choice in the selection of their sites and less likelihood of difficulty with dead spaces in front. For rear guard actions where easy withdrawal is necessary high sites are desirable, but for more determined resistance the low sites are usually preferred if the field of fire is satisfactory.

ARTILLERY DISPOSITION.

The greatest change in entrenched lines in recent years has occurred in the artillery disposition due to the perfection in firing by indirect laying. It is no longer considered necessary or even desirable to bring guns into the foremost infantry line, as concealment and easy withdrawal are both more important than direct fire. To silence the enemy's artillery is one of the necessary objects of the artilleryman, so that batteries may always be expected to draw the hostile artillery fire. No batteries should, therefore, be located near reserves, supports, and infantry lines. It is seldom that artillery will need much artificial cover and its best protection

is its concealment and mobility. Artillery positions are usually somewhat in rear of the entrenched lines where they may be screened from view by natural cover from which they may be easily removed to another site as soon as the enemy has found the range. Many alternative sites are previously selected and ordinarily dispersed sufficiently to furnish a well concentrated cross fire on each possible line of advance of the enemy's infantry. Dispersion is needed to minimize the effect of hits, if the location of the battery is found. Light field guns should be taken to whichever previously selected site will give the best effect of their fire, without special attention being given to the rules formerly followed requiring that their positions should be on the flanks. It is now the rule rather than the exception to fire over the infantry lines. This affords to the artillery a much wider choice of position than formerly. Heavy artillery will conform more or less to the same principles, but the greater difficulty in their removal will limit their mobility and require them to remain longer in prepared positions which will often require much more extensive artificial cover.

It is the general rule of the French Regulations that every field artillery position should be furnished with trenches, whether the guns are hidden or not, and that additional sites for direct fire should always be prepared in advance for possible use. The heavy artillery will usually all be used in the entrenched position without any in reserve. The purpose of this is to cover as much of the front with its fire as possible in order to check the enemy's infantry advance and prevent his batteries from being erected within range. Some of the light artillery, however, in each section will be with the section reserve and the remainder will be available for whatever site it may occupy to the best advantage. Artillery positions will usually be chosen by the chief artillery officer as soon as he learns the commander's general requirements. Artillery will build its own entrenchments and supply its own wire communications.

SINGLE DEFENSIVE LINE.

Some question has arisen as to whether a single line is best on which all the defensive means are concentrated, or whether advanced positions well entrenched should be adopted with successive lines in rear on which the troops may rally if driven out of the first.

The Russians in their recent war adopted almost everywhere

the latter plan. They placed points of support in the form of redoubts in rear of the entrenched outer line and expected to fall back on these points of support when driven out of the advanced positions. Frequently this was accompanied by great loss at the critical moment of leaving the trenches, and the force when it reached the retired position was sometimes so badly shaken as not to be worth much for the defense. Many cases are reported, however, where the Russians' advanced position held out for days and an orderly retreat to the principal position was effected, where a good defense was then made. But Tchabel, a Russian engineer officer, after reviewing the various cases coming within his experience, declared in favor of the single line, as a principle.

The German Drill Regulations state that "it may be taken as a guiding principle to have but one line which may be strengthened by all possible means," although in another paragraph they say, "behind threatened points and behind those where the terrain is favorable, special points of support may be constructed." The Germans prefer the single line mainly as it gives a better view of the tactical movements and affords simpler control of their units for a combined effort.

The French Regulations express a preference for advanced positions.

At Liao Yang three successive lines of defense were built by the Russians, not including the line of advanced outposts. When attacked by the Japanese there was a desperate struggle for the outposts only, and when these were captured the main position was promptly sacrificed.

The Japanese entrenched lines were not often tested severely, but they usually adopted simple, long, fairly straight lines with echelons on the flanks and only occasionally a redoubt or rallying position.

There seems some doubt whether troops in modern war will stand the strain of battle several times in succession, as would be required by several lines. If the first line were not weakened by the troops required to hold the second line, and if the knowledge of this second line were not constantly in mind, the first line might be much more strongly held and a more determined resistance made.

The weight of opinion based on the most recent war experience inclines toward the single line for the usual case. Even the Russians, although favoring several lines, finally moved their support-

ing points nearer and nearer the first lines as the war progressed to get a better field of fire and unite their efforts in defense.

The entire organization of a defensive line, the division of the force into supports and reserves and the arrangements of the communications in rear, are all based on the utmost effort being exerted on the single line selected and any other plan will likely sacrifice part of the defender's strength by interfering with its full development. This should not be understood to imply that advanced positions are never to be occupied, for cases may arise when such occupation would be plainly indicated as necessary. Such advanced positions in general should be so situated that they may be supported from the main line, be under its fire if captured and be readily abandoned without undue loss when no longer tenable. Nor is the single line theory to be understood as preventing a commander from selecting positions in rear where his general reserves may form or where a more protracted defense may be made should his main line be pierced, but the preparation of such sites is better left until their necessity and location are plainly indicated by the progress of the action.

2d Part—Entrenched Lines.

In this country where our immediate neighbors are numerically weak, it is difficult to see any good reason for constructing defenses against a land force long in advance of hostilities. Practically all of our measures of this kind must be improvised. Hastily built trenches may ultimately grow into works of great strength if the emergency should require and the time be available.

Reference has already been made to the use of the shelter trenches for the offensive. Although their general location will usually be designated by some one in authority, the actual location of each man's trench must often be decided upon by himself and he must be taught and relied on to move forward or back until he secures a full field of fire.

It is in the location of the fire trenches of an occupied defensive line that more care is possible, and an analysis of the essentials of a good line will be more useful than remembering rules.

To begin with, the line when chosen on varied ground will be found to consist of a series of points of some natural strength on which the line will rest. These points will usually be knolls, elevated ridges or forward slopes where positions may be found giving a good fire control over the front or where some obstacle such as a

swamp or creek may make it difficult for an enemy to attack without considerable exposure. Each point of this kind may be called a supporting point or pivot for want of a better name and will ordinarily be defended from a group of trenches so placed as to cover the front and flank without having any dead spaces. They will invariably disturb as little as possible the natural appearance of the ground.

In these supporting points there may be placed machine guns for flank protection and if time allows and the conditions require it, overhead cover for some of the troops.

The trenches will be short, seldom longer than a company front, laid out on contours and connected with each other and with the rear by concealed paths or trenches. Supports will be concealed behind a knoll or ridge and blindages, covered trenches, or other protection will be constructed for them if found necessary. The normal force for such a supporting point would ordinarily be about two companies, of which one company or perhaps only one platoon would be constantly in the trenches and the remainder in support, but trench space to develop the fire of all the garrison with some additional space for the reserves should ordinarily be prepared and ready.

Each supporting point in the entrenched line should dominate the surrounding area to the front and flank up to the limit of its effective fire. If the surrounding area is open and without cover for the enemy, the width of this surrounding zone might be from 1,000 yards to 1,250 yards, whereas it might otherwise be as small as 500 yards or perhaps even less if the country were much cut up. If two adjacent supporting points are so placed that their areas of fire overlap, there will obviously be no point through which an attack can be made that will not be under fire. This interval between supporting points may thus vary from 1,000 to 2,500 yards, depending upon the ground. The intervals between will be further protected by ordinary trenches, and the entire line will thus consist of a series of small groups constituting pivotal points and intervals with trenches mutually supporting each other, each covering its own front and giving a cross fire over a part of the line.

These natural points of support were formerly defended by redoubts or other closed works and this means of defense was at first followed by the Russians in their late war. They were often effective, but usually presented conspicuous targets disclosing their

positions and were often congested with men and artillery, thus increasing their losses. More effective curved fire on the part of the Japanese would have made them even worse shell traps.

The German Regulations limit the use of closed works to a few cases, such as where an isolated point is to be held by a few troops or is subject to surprise or where the use of artillery is improbable. In the ordinary case a supporting pivot composed of a group of trenches is preferred in their regulations.

The English Field Manual likewise advocates a group of trenches for a supporting point in the front line. This manual states that redoubts or closed works should never be built on sites where they will be recognized as redoubts by the enemy. This limits their use to somewhat retired positions where they may be concealed.

An English writer describes the modern design of supporting point as a redoubt taken to pieces and the parts so distributed as to afford the best fire with the most concealment.

It may be said, however, that in broad, flat country redoubts or closed works may sometimes be necessary and that in these cases the plan may approach a type design, but even these works should be as inconspicuous as possible, thus requiring a low relief above the ordinary surface.

GENERAL LOCATION.

The general location of the line of works is decided by the commander in accordance with his mission. It must be so placed as to keep the enemy's artillery at a sufficient distance to prevent effective bombardment of the towns, docks, stores, naval bases, railroad terminals, bridges or whatever centers he may be ordered to protect. If he is only expected to block an enemy's advance, the nucleus may be his own base or his stores of supplies. If aimed and observed fire is possible to the enemy, the line should be situated so as to keep him about 10,000 yards away. If such aimed fire and observation of hits are not possible, the line may be brought in closer and thus shortened.

TRENCH PROFILES.

In examining the kind of trench that is adaptable to our purposes, we notice many new tendencies. The high parapets and broad trenches of a few years ago have given way to the deep and narrow trenches with low parapets. This might be expected from the improvements in rifle and field artillery fire, but the Russo-

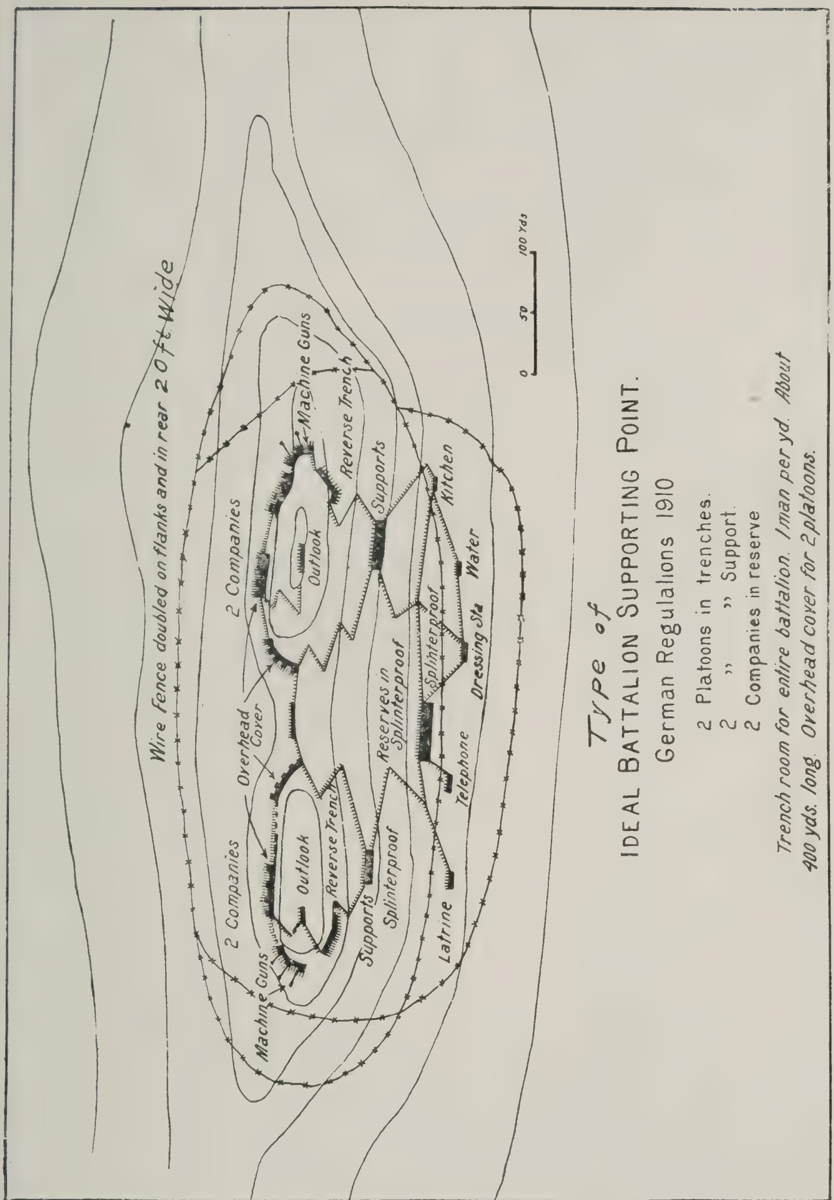

Wire Fence doubled on flanks and in rear 20 ft. Wide
2 Companies
2 Companies
Machine Guns
Machine Guns
Overhead Cover
Outlook
Outlook
Reverse Trench
Reverse Trench
Supports
Supports
Splinterproof
Splinterproof
Reserves in Splinterproof
Kitchen
Water
Dressing Sta
Telephone
Latrine
0 50 100 Yds
Type of
IDEAL BATTALION SUPPORTING POINT.
German Regulations 1910
2 Platoons in trenches.
2 " Support.
2 Companies in reserve
Trench room for entire battalion. 1 man per yd. About 400 yds. long. Overhead cover for 2 platoons.

Fig. 3.

Japanese War has pointed out many other improvements in these matters as well as in obstacles, overhead cover, hand grenades, searchlights, and other accessories. In the beginning the Russians sought to content themselves with kneeling trenches to save labor, but were forced to adopt standing trenches before long to reduce their losses. The German Regulations state that only lack of time will justify kneeling trenches and that standing trenches ordinarily should be provided. The height of the Russian trench from bottom to crest was usually 2 meters (about $6\frac{1}{2}$ feet) and the German Drill Regulations specify 1.9 meters (about 6 feet 3 inches); Japanese, 5 feet 7 inches.

In their recent war the Russians commenced by using trenches cut entirely in excavation, making it necessary to carry away all the earth removed, but they found that on distant slopes the lights and shades made these trenches quite as visible as those with parapets and that the additional work was not justified.

The field of fire and shape of the ground and its vegetation will usually of itself determine where the crest line should be placed, whether raised above or lowered to the level of the ground. Usually the crest of the parapet should not be over about 18 inches above the ground and the top surface should be a single, gentle slope, as both of these conditions favor invisibility. The inside wall of the trench should be as nearly vertical as possible as this provides better shelter, and an elbow rest about 9 or 10 inches below the crest and about 18 inches wide is helpful to the soldier while firing.

A trench narrower than 2 feet can not be conveniently dug and used, and this is too narrow for two men to pass in or for a man to sit in with his back to the parapet; 3 feet is the minimum for this purpose. A height of 4 feet 3 inches or 4 feet 6 inches from the bottom of the trench to the crest of the parapet is necessary for a man firing while standing.

If a trench 2 feet wide and 3 feet deep were dug with vertical sides and a parapet 18 inches high with a slope to the front of 1 on 6 were made of the excavated earth, we would have the simplest trench a man can conveniently fire from, but when not firing he would have to crouch down to be protected or sit facing the end of the trench. In soft earth this trench will require one hour's work for 5-foot tasks. Widening the trench to 3 feet will take another hour, but will enable men to pass in the trench and permit the defender to sit with his back to the enemy. If now we deepen the rear part of the trench 2 feet more, taking about one to one

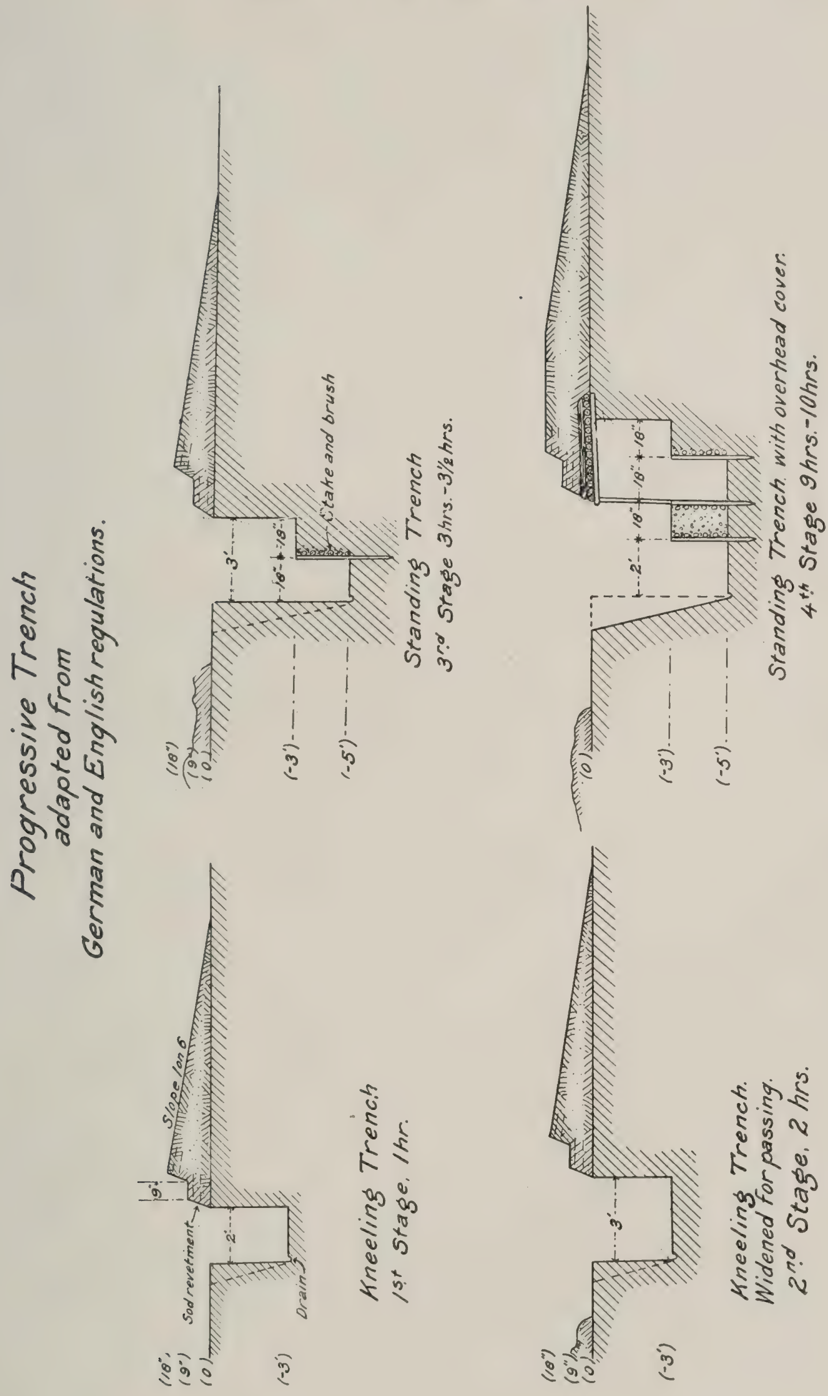

Progressive Trench
adapted from
German and English regulations.
Kneeling Trench
1st Stage, 1 hr.
Slope 1 on 6
9"
Sod revetment
2'
Drain
(18")
(9")
(0)
(-3')
Kneeling Trench.
Widened for passing.
2nd Stage, 2 hrs.
3'
Standing Trench
3rd Stage 3hrs.-3½hrs.
Stake and brush
18"
(-5')
Standing Trench with overhead cover.
4th Stage 9hrs.-10hrs.

Fig. 4.

and a half hours more, a good trench will result, sufficiently deep to protect a man walking erect, and by using brush and poles for a support for a covering of earth and by widening the trench backward, it may be converted into a standing trench with overhead cover in about nine to ten hours in all. To better fix the necessary dimensions in memory it will usually be sufficient to remember the size of the simplest trench, namely, about hip deep with vertical sides and wide enough to sit in. The parapet should be only high enough to conveniently fire over. With this as a basis, all the remaining development will suggest itself from the necessities presented at the time. For better communication in the trench and to sit with one's back to the parapet some widening will be necessary, and to afford protection to a man walking erect to the rear the trench must be deepened to bring the soldier's head below the parapet. The soldier can usually be relied on to make these improvements with the least labor.

Revetments when required may be made of fence plank, sand bags, barrels, brush or anything of the sort that is at hand. We may almost always expect to get sod in open country and brush in wooded places, so that some revetment may nearly always be made easily with materials at hand. Sod makes a fair revetment when cut and laid grass-side down, and stakes to hold up a layer of brush will often do as well as more elaborate work. Gabions and fascines are no longer in use, except in some of the more deliberate siege operations.

The essentials of the profile may then be stated to be: First, a low bullet-proof parapet not over about 18 inches high, with a single, gentle outer slope, not steeper than 1 on 6 and an elbow rest 9 inches below the crest; second, a narrow trench with vertical or nearly vertical sides and of sufficient depth to cover a man standing, and wide enough to sit in and permit another to pass; third, some protection from enfilade or oblique fire by using traverses or special arrangement of the ground plan; and fourth, a bottom slope with a gutter for drainage.

OVERHEAD COVER.

If overhead cover is to be provided, it should ordinarily be under the front parapet rather than somewhere in rear. This serves to keep the men near their defensive lines where they can quickly move to their firing position. All projections above the parapet that would disclose the location must be avoided, as well as all

breaks in the crest that reduce the number of rifles and increase work. A design should be adopted that will permit the work already done on the simpler trenches to become a part of the new plan without waste of time and labor. Overhead cover for as many men as practicable will be found very desirable if time allows. It need not be very heavy. Splinter proofs in the trenches for all the troops was the practice in Manchuria.

LOOPHOLES.

Some form of head cover, such as loopholes, notches in the crest, head-logs, or sod embrasures, is advocated by many eminent soldiers as helping to steady the men, increase the accuracy of their fire and reduce losses. Others, however, have held that they reduce the number of rifles, restrict the fire to a frontal direction and disclose to the enemy's artillery and infantry the location of the trenches, thus defeating a part of their purpose. Firing through a visible loophole is more dangerous than firing from an invisible trench. There will undoubtedly occur many places, however, where head cover or at least concealment of heads during firing can be used to advantage, and the local conditions will usually settle this question one way or the other.

The Japanese sometimes used piles of mud between which they fired. The Germans think loopholes and head cover unnecessary in field fortifications, whereas the English usually favor them.

A satisfactory solution of this question would apparently be to place sand bags in the trench, to be placed on the parapet as "headers" having openings for fire should they be found a help.

DRAINAGE.

Drainage of trenches is important. The floor should slope to the rear and a gutter be cut along the rear face to convey water to one end of the trench where a soak pit will usually serve all ordinary requirements. If better drainage is necessary, an outlet to lower ground may be cut and filled with broken stone, irregular limbs or a box culvert before being covered over. If drainage ditches are started first, interruption from rain will be less troublesome.

OBSTACLES.

In every line subject to persistent attack some form of artificial obstacle is desirable in connection with a cleared front. Should

the attack of the enemy's infantry reach a point about 200 yards from the parapet, a crisis would be reached and, if not stopped there by superior fire, the position would often have to be abandoned.

Clearing for a better field of fire should begin at the immediate front and be continued outward as far as time permits up to 600 or 800 yards. It is surprising how little judicious clearing will often provide a good field of fire, but above all it should be remembered that clearing should not be overdone if concealment is not to be sacrificed. A natural screen before a fortified line is of great advantage. Many trees and crops in front should be left standing, as they afford more help to the defense than hindrance. Indeed, large trees if felled may afford more protection to the enemy than to the defenders.

Various types of obstacles have been suggested, but the one offering the greatest advantages is the high barbed-wire entanglement, low enough to stop crawling men. Its effectiveness, its cheapness, difficulty of removal, rapidity of construction and the little interference with the view make it far the best. It is usually built on posts from 4 to 6 feet high set at random, the wires strung irregularly and the whole set in a strip 20 to 30 feet wide around the work to be covered at a distance of 100 to 200 yards. A low wire entanglement was often used in Manchuria by the Russians placed at a distance of about 50 yards to the front, far enough to prevent a hand grenade being thrown into the trenches, and often covered from sight and artillery fire by a parapet about 2 feet high. In special places, it may be found desirable to have two rows of wire entanglement around a work.

Military pits were also used extensively in Manchuria, there being but little timber on hand for other kinds of obstacles. They consisted of holes about 5 feet in diameter at the top, about 5 feet deep, 2 feet in diameter at bottom with a sharpened stake at the bottom. They required too much labor to construct for frequent use. The earth excavated was used in making a cover or glacis for it. These pits were often covered with barbed-wire entanglement in addition. Abattis and slashings will be often used where timber abounds, especially if much clearing is necessary.

All obstacles take time and must be built with the means available, but if the position is to be occupied for some time, they may easily be worth what they cost in time and labor by preventing surprise, delaying night attacks, but especially in breaking up the

enemy's formation and confining him to certain lines of approach.

Many other accessories, such as dummy trenches and dummy batteries, hand grenades, land mines, fougasses, etc., are often used, but scarcely come within the average officer's experience. They are special devices that may always be found fully discussed in text-books and are only used when time is ample. Searchlights have shown their usefulness and will likely be more necessary in the future, as night attacks may be expected to become more common.

EXECUTION OF ENTRENCHMENTS.

The execution of entrenchments presents some difficulties, for sufficient tools and time will often be lacking and system must be sought in the arrangement of working parties to get the best results. The tactical situation demands that a certain front be entrenched by a given number of men with a given number of tools in a given time. What form of trench is the best? To solve this question we must know the work capacity of our men. A soldier untrained in digging can not remove much over 80 cubic feet of ordinary earth in one continuous period of work, whatever its duration. He can remove 30 cubic feet in the first hour, but his average for 4 hours is about 20 cubic feet per hour and 4 hours is about the limit of his endurance. This constitutes what is technically called a relief. If tools are double-manned, an increase of one-third the quantity is obtained. If we call the average output of one man 20 cubic feet per hour, then by double-manning the same set of tools 27 cubic feet may be done. This is in easy earth where the throw is not over 12 feet and lift not over 4 feet. Men can not work closer than 5 feet without crowding, but if necessary they may be crowded into 4-foot distances, except at night. Five feet is the usual interval, but it is sometimes increased when sufficient tools are lacking.

The tools available for digging will usually be only those allowed by regulations. The light portable entrenching tools will not be ordinarily used for this kind of work.

To determine the best kind of trench that can be made under ordinary field conditions, we must make a hasty calculation based on the cubic contents of the trench and our work power. The output of each shovel is 20 cubic feet per hour or, by double-manning, 27 cubic feet per hour. Suppose we had a brigade of three regiments on the line and all available for work. If each company had about 150 men and if we should have, say, three hours with only

their authorized allowance of tools, what sort of a trench would be dug? In three hours each shovel, if double-manned, should excavate 81 cubic feet or three times 27 cubic feet. The length of the company trench will have to be about 150 yards, or 450 feet, for a company of 150 men, and this divided by 48, the number of tools to be assigned to each company under orders, will give 9 as the length of each task; 81 cubic feet being the cubic contents of the work required of each shovel and 9 feet its length, the cross section would have to be 81 divided by 9 or 9 square feet. Therefore, a trench 3 feet deep and 3 feet wide would then be the best that could be done. For this work we would probably detail 96 men in each company as diggers with about 10 men with picks and 10 as reserves; of the remainder about 10 with the pick mattocks will be directed to shape the parapets and 20 with axes will clear the front. The remaining will be available for police, guard, patrols, or other military duty.

Troops detailed for trench work will be marched to the line to be fortified in a single file, take up their tools as they pass the piles, where placed by the engineers, assume the proper intervals along the line and when instructed as to how they will work will commence operations. The engineer troops will be called on to superintend the details of the work.

In connection with this subject it appears that there are several considerations that can well be emphasized. Every officer who has field fortification to execute may be expected to find that sufficient time is lacking, sufficient tools and often sufficient men, but he must obtain his results with the means at hand and must first find out therefore where he stands in time, tools, and men. A rough calculation will save time and confusion in the end. Tactical requirements may come next and later the details. War gives no opportunities for working out fine-spun theories; the all important thing is to get something done. Rules and books are after all only guides to be applied with an admixture of common sense, and it is always wise to have a good margin over their figures for safety. It is always best to keep to units in construction, administration and defense of entrenched lines. This tends toward simplicity, avoids much confusion, and makes it possible to trust many things to the common sense of other officers. Cover should be sought first in actual construction, then a cleared field of fire and, later, obstacles.

Rock Drilling, Tuscumbia Bar, Tennessee River

BY

Mr. J. E. Hall
Assistant Engineer

This obstruction to navigation is 211 miles below Chattanooga and 253 miles above the confluence of the Tennessee River with the Ohio, at Paducah. It begins at a point about 3 miles below the Florence bridge, abreast of the Sheffield power plant, and extends approximately 2 miles down the river. Its relative position is shown on locality sketch herewith.

This shoal is composed of a series of blue flint-limestone ledges, which are overlaid with a thin coating of gravel. The ledges are about level transversely, but have a considerable downstream dip.

The project for improving this place contemplates excavating a channel near the south shore, 150 feet wide and 5 feet deep at extreme low water, from deep water above to deep water below the bar. The material to be moved being very hard flint rock, it was necessary to drill and blast it before it could be dredged.

It was originally intended to contract this work provided a favorable offer was received, and it was not until after the proposals were opened about June 1, 1911, that the decision was made to do the work with hired labor.

The work here being the first rock excavation where drilling and blasting was necessary, on this section of the river, we were at an experimental stage in regard to plant and methods. A small amount of drilling had been done on the upper division (above Chattanooga) where rafts were used for carrying the drills, and a derrick boat was used with each raft for a tender, the derrick being necessary in moving since there was not enough buoyancy in the solid timbers of the rafts to float the drills when the spuds were raised. Extensive drilling operations being necessary here in rock where the depth of water varied between a few inches and 5 feet at low water, it was thought advisable to design floats with sufficient buoyancy to carry the drills, men and material necessary

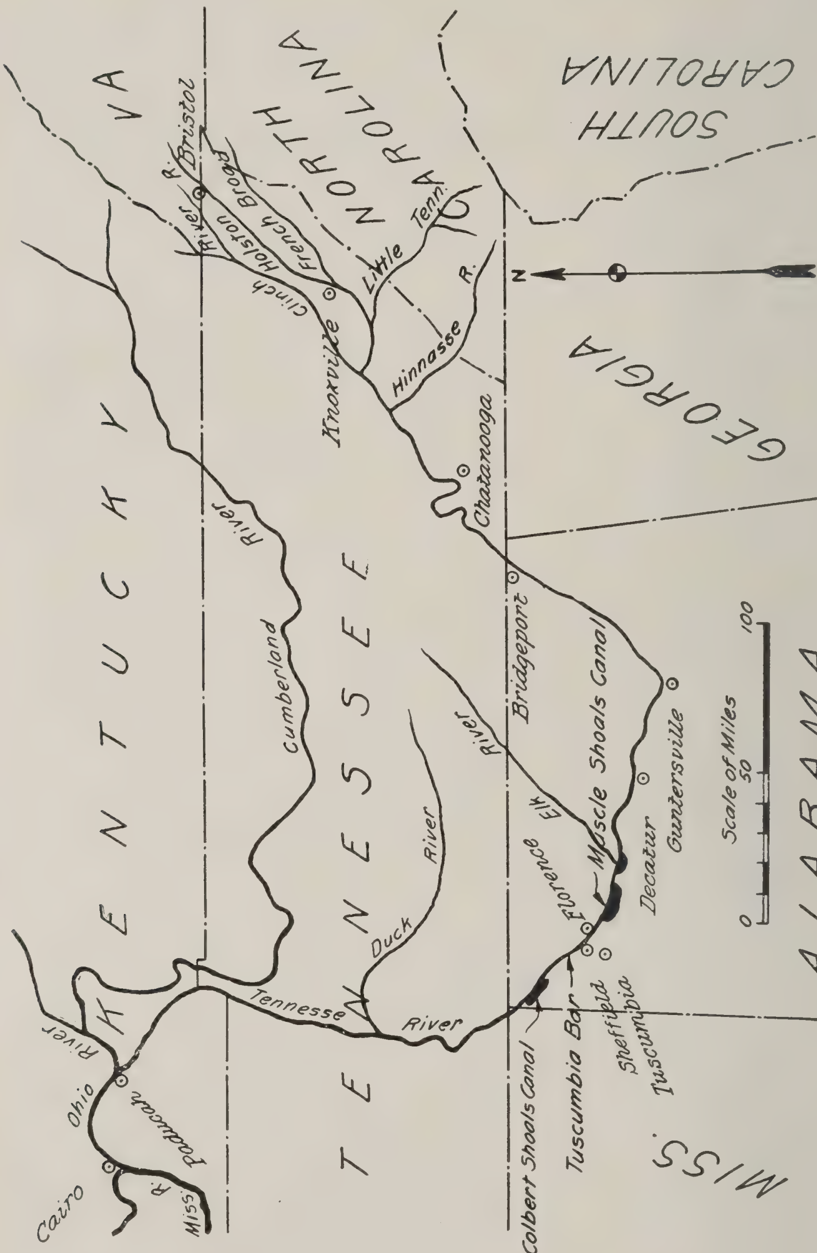


Fig. 1. Vicinity map, showing location of Tuscumbia Bar in Tennessee River, northwestern Alabama.

to operate them, and to also keep them as light as possible in draft, as it was necessary to pass them over some very shallow water.

When it was definitely decided that this work would be done by the United States, requisitions were promptly submitted for drills, material, lumber, etc., necessary for building the floats. The first shipment of ten drills was received August 15, and a second lot September 20. These were put in commission as quickly as practicable after being received, and on September 21 three drill units were in operation, each mounting six drills.

The floats used for the first season's drilling (1911) were composed of nine small boats, the dimensions of each being: depth, 1

Fig. 2. View showing drill units at work; also portion of channel after being blasted, looking down river. Gauge reading at time of exposure for picture, 2.7.

foot; width, 5 feet, and length, 25 feet. These were arranged in three rows with three boats in each row and a space of 2 feet was left between the lines of boats through which to operate the drills. During the first season they were turned longitudinally across the current and the float held in place with spuds which were 6 inches square. The dimensions of the floats when the boats were so arranged were 19 by 75 feet. At each placing of the drill unit two lines of holes were drilled extending one-half the way across the channel and twelve holes put down in each line. These lines being 7 feet apart made the average spacing of the holes 6¼ feet by 7

feet, and the holes were put down to a depth of 7 feet below low water.

The drilling was carried on on this basis until December 15, 1911, when it was terminated by high water and bad weather. During this season's work about 2,500 linear feet of channel, beginning at the upper end of the work, was drilled and blasted. This was over the lightest part of the drilling, as it began at grade point and the depth decreased very gradually, so that the depth of the holes varied from 2 to 6 feet in the rock.

Subsequent dredging has proven that this spacing of the holes was too wide for this class of material and not deep enough for properly loosening it up. Where the detonation was perfect the rock was usually broken up, but often in such large pieces that they were very difficult to handle, and it was sometimes necessary to break them again with mud capped shots before they could be handled, but when any of the holes failed to fire an unbroken area was left which was necessary to redrill. The best results both in drilling and blasting were gotten at a low stage of the river, the effect of a rise being noticeable when a 2-foot stage was reached. The increased current due to the rise made it hard to hold the floats in place and the difficulty of detonation was also increased, as the current and drift would often break the connections.

The following table shows this season's work, giving the unit cost of drilling for each month, also the stage of the river, depth of water over the rock drilled and the depth of the drilling in the rock.

Date.	Linear feet.	Average depth of holes.	Gauge.	Depth of water over rock.	Cost.	Unit Cost.
1911.		*Feet.*	*Feet.*	*Feet.*		
August	3,520	4 to 5	.4 to 2	3 to 5	$1,336.60	$0.38
September	6,338	3 to 5	.3 to 1.5	3 to 4	2,871.20	.40
October	6,921	3 to 5	1 to 5	3 to 7	3,450.00	.50
November	10,373	4 to 5	1 to 3	2 to 6	4,634.39	.43
December	1,679	4 to 6	2 to 12	4 to 7	1,007.40	.66
Total	28,831				$13,299.59	$0.46

Average unit cost for season 1911, $0.46. Number of linear feet of channel drilled, 2,500.

The small progress made in drilling during the month of August was due to the late start and limited number of drills, which also affected the work in September. The October work was also retarded by a rise which caused a week's suspension. There was

only a few days in December when the river was low enough for good results, but the force engaged in drilling was held together until the 15th, on which date they were disbanded owing to high water and bad weather.

During the winter and spring dredging was carried on over the drilled area, affording an excellent opportunity to study the spacing

Fig. 3 (upper). View showing arrangement of drill float, with drills in operation; also portion of blasted channel looking up river. Gage reading at time of exposure for picture, 2.7.

Fig. 4 (lower). Looking upstream over channel after it is drilled and blasted. Drill raft in foreground. In background are dipper and orange-peel dredges removing rock.

of the drilling and the depth. A great many small areas of unbroken material were found which were probably due to failure of

charges to fire, but in some instances where the detonation seemed complete points were left between the holes which the dredges could not reduce to grade. These unbroken areas and high points showed conclusively that the spacing of the drilling was too wide and holes too shallow.

Extensive repairs being necessary to the floats before entering on another season's work, it was decided to rearrange the boats composing them, turning them lengthways with the current and leave the opening between the boats 1 foot wide. The floats used in the season of 1912 were composed of thirteen boats, dimensions 1 by 5 by 25 feet long, put together in this way making the spacing of the lines 6 feet apart instead of 7 as used the previous year. The dimensions of this float were 25 by 77 feet. (See accompanying illustrations, which give a good idea of their construction and also show the appearance of a section of channel after being blasted.)

At each placing of the drill unit or float, twelve lines of holes were drilled, each line having six holes. As the length of the boat was 25 feet, the spacing of the holes was made 4 by 6 feet and the holes put down to a depth of 9 feet below low water. In addition to the advantage of narrowing up the spacing, boats being placed lengthways with the current was an advantage, in that they were less affected by the action of the current and drilling could be carried on at a higher stage of water.

Excessive rainfall during the spring prevented our resuming the drilling until June 15, and very materially interfered with the progress during July. After July, the work was continued without interruption until the latter part of December when it was terminated for the season by high water.

The following table shows the season's work during the season of 1912, giving unit cost for each month, stage of river, depth of water over rock, and depth of holes drilled:

Date.	Linea feet.	Average depth of holes.	Gauge.	Depth of water over rock.	Cost.	Unit Cost.
1912.		*Feet.*	*Feet.*	*Feet.*		
June	3,720	7 to 8	3 to 5	3 to 5	$1,523.19	$0.41
July	3,256	7 to 8	3 to 6	3 to 6	1,465.00	.45
August	15,120	7 to 8	2 to 3	2 to 4	5,727.28	.38
September	15,752	6 to 7	1 to 3	1 to 5	6,301.85	.40
October	20,250	5 to 7	1 to 2	2 to 4	9,317.50	.46
November	20,042	5 to 7	.5 to 1.5	3 to 4	10,462.89	.52
December	7,568	5 to 7	1 to 6	3 to 7	6,659.84	.88
Total	85,708				$41,457.55	

Average unit cost for season 1912, $0.483. Number of linear feet of channel drilled, 2,500.

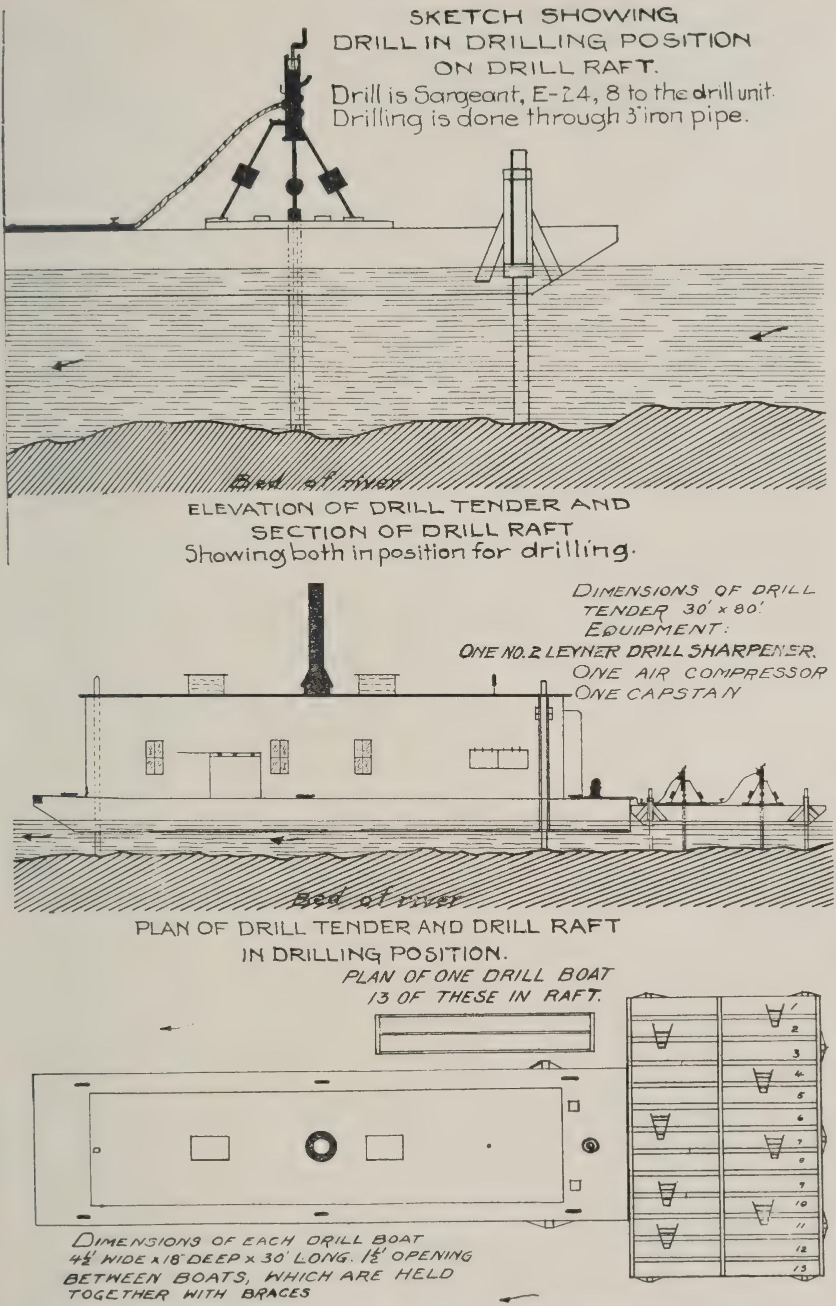


Fig. 5. Plan and elevation of drill tender and sketch showing a drill in position on drill boat.

In comparing this season's work with that of 1911, it will be noted that the unit cost of the drilling is 2 3-10 cents in excess of the 1911 cost. This may be due to considerable advantage in weather and river conditions in 1911, and also to the fact that the drills were all new in 1911 while in 1912, especially near the close of the season's work, a number of them were badly worn and would not deliver a normal stroke. After September the cost is also augmented by the shortening of the days, making it necessary to work a longer number of hours at night. While the plants are very well lighted, quite a falling off is noted in their progress when comparing the result of an hour's work at night with an hour's work in the day. Early in October a number of the best drill men left the work to go back to their old places at the furnace, and it was necessary to fill their places with new men. The November work was affected by two holidays, November 2d and Thanksgiving, and December by high water, which finally terminated the work for the season.

The following method was universally followed during both season's work: The drill floats were placed in proper position for work by accurately lining them with ranges, marking the center of each half of the channel, and by cross ranges which marked the lower extremity of the drilling and blasting already done. A float thus lined is in position for drilling one-half the width of the channel for a distance of 25 feet, or length of the boats. At each setting, twelve lines of holes were drilled with six holes in each line. Each float carried eight drills, all of which are operated by steam.

To prevent the holes from filling with gravel and silt, etc., the drilling was done through tubing or pipe, 3 to 4 inches in diameter. From three to five drill bits were used in putting down each hole, the first bit being 2½ inches in diameter and the last 1¾ inches. When the hole was down to a proper depth, a pipe that would exactly fit the top section of the hole was put in and the bit taken out and the hole loaded, the charge consisting of 80 per-cent gelatin dynamite, and varying from 6 to 10 sticks according to the depth in the rock. The stick having the primer was placed about one-third of the way down from the top, having from 2 to 3 sticks on top of it and 4 to 6 under it. The charge was firmly packed down in the bottom of the hole with wooden poles which fit very closely the section of the hole. When the loading was completed the hole was marked by a cane, which was firmly embedded in the charge, leaving the top of the cane about 6 inches above the surface of the water

and the primer wire looped around the top of the cane. When all the holes were loaded the primers were all carefully connected so as to make 3 circuits, 24 holes to each circuit, leaving the end wires of the first and fourth line looped around the top of the cane so that they might be readily found and connected with the lead. The float was then dropped down from over the holes and a set of lead wires attached to each circuit. When these were connected (insulated tape being used for making all these connections) the float and tender were dropped back about 250 feet below and the charges ignited by using three large batteries simultaneously.

While each of the batteries would fire all these charges on shore, 24 holes was about their limit under water. This method was usually successful in getting off all charges together when the river was at a stage below 3 feet, but above this stage the connections were often interfered with by the force of the current and running drift, etc., necessitating several reconnections with the lead wire in order to get off the charges, and it was frequently the case that the wires were broken or withdrawn from the caps, making it impracticable to fire the charges.

During the first season the drills were all sharpened by hand, but in 1912 a Leyner drill sharpener was installed which was operated with compressed air. This machine proved perfectly satisfactory as it gave the bits more uniform gauge so that there was no diffiiculty of one drill following another, and the bit sharpened by this machine seemed to stand the hard service better than those sharpened by hand. A considerable saving was also effected, as one blacksmith and helper were able to sharpen steel for 24 drills, while three blacksmiths and helpers were necessary to do this work by hand.

The seeming excessive cost of drilling here is due to the character of the rock, which is the very hardest of flint. It was found a very difficult matter to get steel that would stand this rock, and it was often necessary to change the bits several times in getting down one sweep of the drill, which is only 24 inches. The slightest mistake in tempering would cause the bits to fail at once. In this connection it is interesting to note the difference between the rock at this place and the rock at Buck Island Shoals, 3 miles below this place, where rock excavation was carried on at the same time. The rock at Buck Island shoals is soft oolitic limestone which can be drilled easily and rapidly, and the drill bits keep sharp indefinitely. In addition to the ease with which it can be drilled, it is also very easy

to break up. While it was found necessary to space the holes at Tuscumbia Bar 4 feet by 6 feet, a spacing of 7 feet by 8 feet at Buck Island was found ample, this spacing breaking the rock up better and leaving fewer large pieces than at Tuscumbia Bar. The same plant and methods were used at both places. The cost per linear foot at Buck Island Shoals was found to be \$0.25 against \$0.483 at Tuscumbia Bar.

During the drilling season of 1912, 85,708 linear feet of holes were drilled and blasted. It is estimated that each linear foot of drilling loosened up .88 of a cubic yard and that 76,185 cubic yards were made ready for removal by the dredges at a cost of \$0.543 per cubic yard for the drilling and blasting, and ¾ of a pound of dynamite was used for each cubic yard blasted.

When the river was at a favorable stage for drilling it was found that an entire day with a double crew was required to drill out, load and detonate all the charges for one setting of the drill unit. Whenever unfavorable conditions occurred from weather or high water the progress was considerably lessened.

The season's work for 1912 covered the heaviest part of the blasting, as it begun at the upper end of the extremely shallow water and extended entirely below it. While the number of linear feet of holes drilled in 1912 was about three times the amount drilled in 1911, it only covered about the same channel area, viz: 2,500 linear feet. This was on account of the narrow spacing of the holes, the additional depth drilled and also that this season's drilling covered the heaviest part of the work. There is now left above the dam an area undrilled about equivalent to 1,200 linear feet of channel.

	Linear feet.
Average day's work for one drill unit operated with a double crew	432
Average hour's work	27
Average hour for one drill	$3\frac{3}{10}$

The drill unit used for work here consists of a float for carrying the drills, and a boat of some type having a boiler with sufficient power for furnishing steam for all of the drills. The floats have been modified to some extent, it being thought best to deck them, since with the decks they are less liable to sink during the heavy wind storms which we sometimes have.

During the first season's work any boat having sufficient boiler power that could be spared from the plant was used as tender. The floats could be very quickly built and they were put to work in this way pending the building of a suitable tender. The type of boat built for this purpose is a barge 30 by 80 by 4 feet in depth, pro-

vided with three spuds. It is equipped with a 90-horsepower boiler, one Leyner drill sharpener, one compressor for operating same, and a steam capstan for handling the barge. The drill sharpener is

Fig. 6 (upper). Showing a 1½-cubic yard and a 2-cubic yard bucket, each of which failed by tearing away of lip; manganese steel lips and teeth.

Fig. 7 (lower). Dipper dredge *Tuscumbia*. Hull, 100 by 34 by 6 feet 10 inches; main engines, 10 by 14 inches; bucket, 1½-cubic yard for rock, and 2 cubic yards for gravel. Cost, $44,000, complete.

Leyner No. 2, and the sizes of steel for forming the bits are 1¼,

$1\frac{3}{8}$, and $1\frac{1}{2}$ inches. The sharpener is driven by air, the compressor in use being one manufactured by Chicago Pneumatic Tool Co., having a cylinder 9 inches for steam and air, by 11-inch piston stroke, piston displacement 130 cu. ft. air per minute at 100 pounds pressure. This tender was built here and equipped for the work, the cost being as follows:

Building hull, cost of labor and subsistence	$1,346.66
Lumber and iron, nails, spikes, oakum, etc.	1,902.40
One 90-horsepower Brownell boiler	900.00
One Leyner drill sharpener, No. 2	697.10
One compressor for running same	533.00
One receiver for air storage	64.80
One steam capstan	495.00
Setting up and connecting above	185.40
Building one-story cabin for sheltering machinery, material, and labor	465.00
Total cost of tender ready for use	$6,589.36

Type of float now in use, composed of 13 boats; cost, labor and material	$1,140.00
Eight drills, E-24 Ingersoll-Sergeant	1,950.00
Steam hose, drill steel, iron pipe	365.00
Cost of float, equipped for drilling	3,455.00
Cost of tender	6,589.36
Cost of one drill unit	$10,044.36

Three drill units were operated during the season of 1912, and it was found that one drill sharpener could keep steel in shape for the three units, each of which carried eight drills. Only one tender, as described above, has been built for the work here, the other two units being furnished with steam by spare pieces from the plant. It was found that the greatest wear and deterioration in the drill units is in the float and the drills, it having been found necessary to rebore and overhaul a number of the latter and that two seasons' work is about all that the floats will stand.

The area drilled in 1911 is shown on accompanying sketch in solid black, extending from the upper end 2,500 feet downstream. Estimated in place there are 35,136 cubic yards of material to be removed in order to reduce this area to grade. As previously stated, when this area was dredged a great many high points were found, making it necessary to reblast a considerable portion of this channel, and suggesting the advisability of deeper and closer drilling.

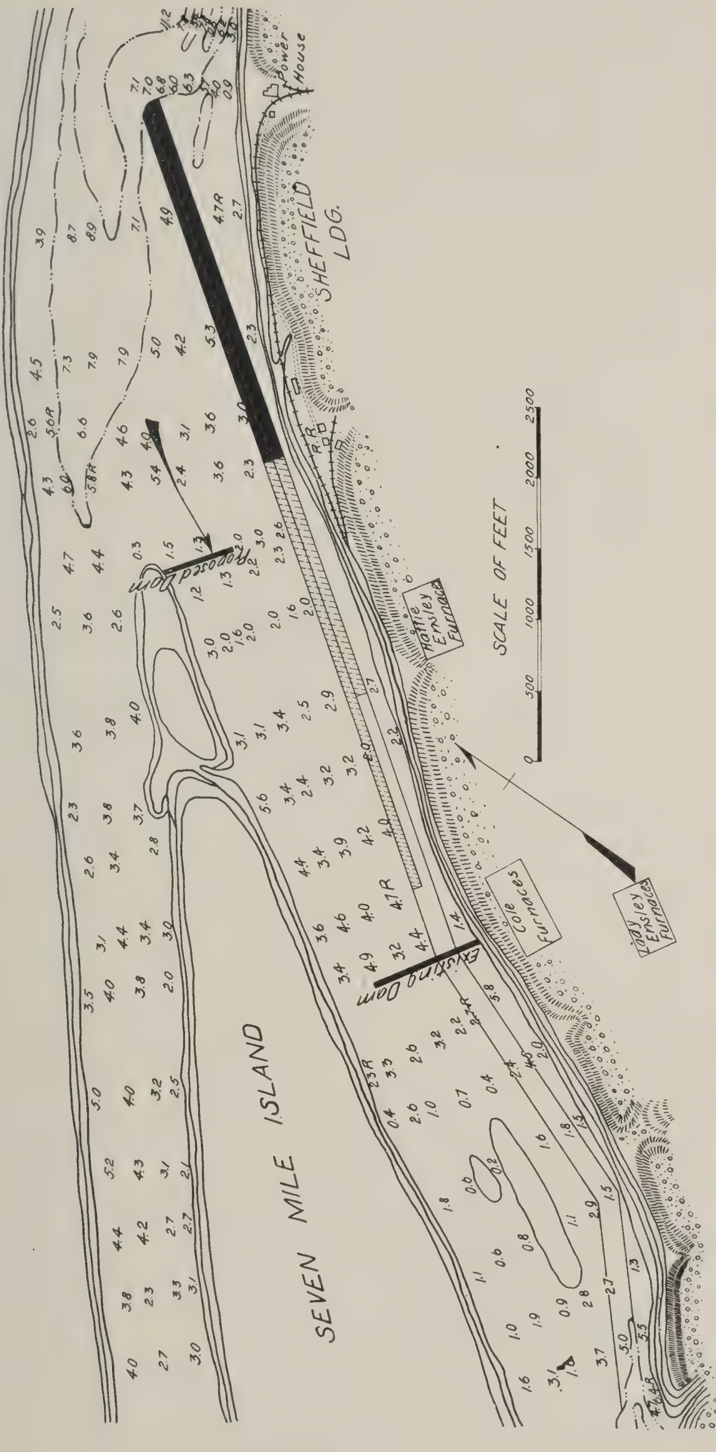


Fig. 8. Tuscumbia Bar. Solid black shows portion of channel drilled in 1911 and since dredged. Hatched area shows portion of channel drilled in 1912, on which the dredges are now working.

During this season 28,831 linear feet of holes were put down, loosening up about 23,155 cubic yards of material, costing as follows:

Actual field cost, including material, salaries, and subsistence, etc.	$13,299.59
Deterioration of plant on account of season's work	3,840.00
Overhead charges	664.98
Total cost	$17,804.57
Cost per linear foot	0.617
Cost per cubic yard, loosened	0.76

The hatched area on sketch shows portion of channel drilled in 1912, on which the dredges are now working. The closer spacing of the holes, 4 by 6 feet, has served to loosen up the material much better and the dredges are able to get "grade," with the exception of a small area which was drilled when the river was too high. During the season of 1912 85,708 linear feet of holes were drilled and blasted, loosening up 76,185 cubic yards of material estimated in place, costing as follows:

Entire field cost, material, etc.	$41,457.75
Estimated deterioration of plant	7,500.00
Overhead charges	2,072.88
Total cost	$51,030.63
Cost per linear foot, drilled and blasted	.595
Cost per cubic yard, loosened	.677
Total amount of material loosened up in the two seasons 99,566 cubic yards.	
Total amount of dynamite used, 75,450 pounds	$12,855.36
Amount of dynamite per cubic yard, ¾ pound	.127

A Portable Field Girder*

MARK II

BY

Bt. Maj. R. L. McClintock
D. S. O., Royal Engineers

In the July (1912) number of *The Royal Engineers Journal* there appeared an article describing a Portable Field Girder, in the construction and launching of which certain experiments had been carried out at Bangalore by the "Queen Victoria's Own" Sappers and Miners.

FAULTS OF ORIGINAL PATTERN.

The original girder bridge on closer acquaintance, however, appeared capable of improvement in two directions:

a. In longitudinal stiffness;
b. In vertical rigidity.

Any lack of the former quality affects this type of bridge both during the operation of launching and when under a load, causing a tendency to buckle sideways in each case. On the original bridge this tendency was somewhat marked during launching, and had to be neutralized by temporary cross-bracing placed horizontally at the road level. When under a load the buckling tendency was very much less, and the usual wind-ties from each side of the bridge to the banks were sufficient to resist it.

The lack of *vertical* rigidity only affected the bridge during launching, at which time the tie-beams of the girders came into compression and had a tendency to buckle upwards out of the horizontal plane, which, if permitted, would have led to the "shutting up" of the girders. This tendency was met by lashing temporary struts up and down across the girders, as explained in the paragraph "Launching Expedients." Both the horizontal cross-bracing and these struts were removed as soon as the bridge was safely across the gap.

Now, if the bridge were to be truly portable, it was necessary to cart about these temporary additions equally with the other components of the girders. So it seemed that they might as well be incorporated with the system at once—especially if any additional

*Reprinted from *Royal Engineers Journal* for January, 1913.

and permanent rigidity was to be gained thereby. In this it seemed as if they might be substituted for certain other parts of the bridge, such as the main ties OB, MC, MD, and NE (*vide* Fig. 4 of lithograph). These, being made of timber instead of wire, would thus be capable of first acting as struts during launching (and so prevent the rising of the roadway), and later as ties, when the bridge came under a load.

In addition to the above alterations, the experiment was made to see whether a saving of weight could be effected by replacing the double 5 inch by 4 inch compression members AB, CD, EF (*vide* Fig. 1, page 564, "Field Girder Bridges") by single ones 5 inches by 5 inches in section, similar to the rest of the compression members.

MARK II BRIDGE.

On these lines a new 60-foot girder bridge was constructed, which is fully described in the accompanying photographs, plans, and tables. It is calculated for infantry in file and loaded pack mules, or 3¾ cwt. per foot run dead load. The roadway is 6 feet wide, and it is hardly necessary to point out that the construction of a narrow bridge like this is a more severe test of the system than would be one of a similar span and a roadway of normal width.

This improved bridge is known as the "Bangalore Girder, Mark II," and has given entire satisfaction both by its behavior during launching and under its full load. There are now no extras of any sort to be added to the bridge for launching; it is put across the gap absolutely complete, except for the chesses. The system is also quite rigid in every direction under all conditions, and so stiff horizontally that wind-ties are hardly necessary. The gain in individual stiffness of the two girders, due to the above substitution of timber ties for certain of the wire ties, also considerably facilitates the putting together of the bridge prior to launching.

In addition to the alterations already mentioned, the experiment was made of providing each shore transom with two runners (Fig. 11), both to facilitate the rotation of the near shore transom round the foot of the derrick during launching, and to enable the bridge to be moved slightly forward or backward after launching, should this be necessary. They are advantageous, but not essential.

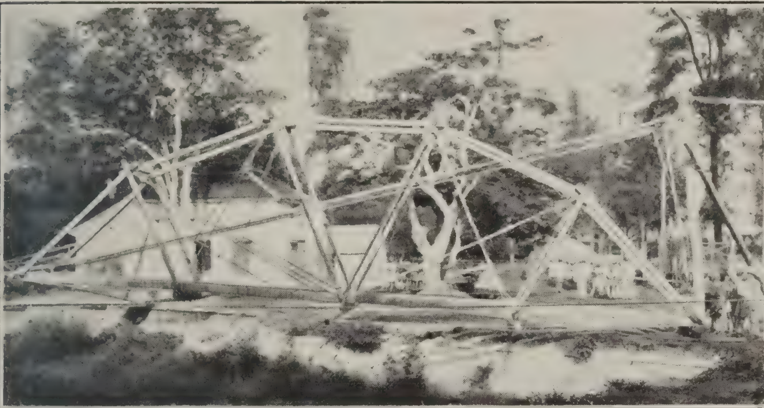
WEIGHT OF COMPLETE BRIDGE.

The increase of launching weight of Mark II over the original bridge is the difference between 5,799 pounds and 5,436 pounds—that is, 363 pounds. The weight of the original bridge as given (5,436 pounds), however, was exclusive of the temporary struts and cross-bracing, which were necessary in launching it. These are now included in the total weight of Mark II (5,799 pounds), and as they represent well over 600 pounds, a distinct saving has been attained by the new design. The whole bridge (Mark II), in-

cluding chesses, can now be carried in eight two-wheeled Indian ox-carts.

LAUNCHING.

The method of launching Mark II is identical with that de-



Swing Commenced.

Far End of Bridge Lowered into Final Position.

Fig. 1.

scribed in the original article, no improvement on that system having been discovered. As an experiment in shortening the derrick as much as possible, the 60-foot bridge (Mark II) was launched complete, except for chesses, with a 20-foot derrick and a $4\frac{1}{2}$-inch tackle, the operation of raising, swinging, and lowering occupying

just five minutes. Convenient trees were used instead of artificial holdfasts for the derrick guys.

DURABILITY.

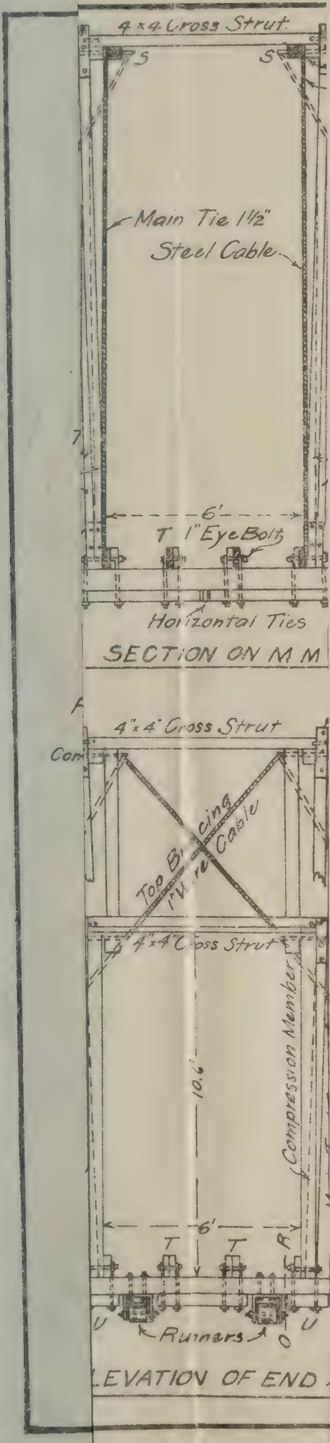
As regards the durability of this type of bridge it is impossible to speak without longer experience. The bridge shown in Fig. 2, however, has been launched, picked up and relaunched several times during the last six months; has been loaded from time to time with various sorts of loads; and has in the intervals stood across the gap shown, enduring an Indian hot-weather and rains without so far showing any signs of decay.

Note.—My thanks are due as before to Regtl. Sergt.-Major King, R. E., and Q. M. S. Instr. Dolan, R. E., of the "Q.V.O." S. & M., who respectively assisted me in the design of the bridge and its launching, and also to Sergt. Short, R. E., who prepared the working drawings.

Weights and Sizes of Component Parts of 60-ft. Bangalore Girder Bridge. Mark II.

Name or position of piece.	Size of section Inches.	No. of pieces.	Weight of each. Lbs.	Total weight. Lbs.
Woodwork.				
Compression members*	5 by 5	10	164	1640
Tie-beams and road-bearers	8 by 2	16	84	†1344
Main ties (timber) long	4 by 4	4	120	480
Main ties (timber) short	4 by 4	4	100	400
Shore transoms	9 by 6	2	150	300
Roadway transoms	8 by 4	3	102	306
Cross struts	4 by 4	4	40	160
Outriggers	4 by 4	6	65	390
Blocks		8	8	64
Ironwork.				
Eye-bolts (S)	$1\frac{1}{4}''$ round	8	10	80
Eye-bolts and brackets (TU)	1" round	20	16	320
Main ties, $1\frac{1}{2}$ inches, long	$1\frac{1}{2}''$ circum.	4	15	60
Main ties, $1\frac{1}{2}$ inches, short	$1\frac{1}{2}''$ circum.	4	10	40
Runners complete with brackets (shore transom)		4	25	100
Top bracing, 1 inch cable	1" circum.	1	25	25
Horizontal bracing, with straining screws	1" circum.	1	30	30
Outrigger bracing, with straining screws	1" circum.	2	30	60
Total				5,799

*Timber: "Honnee" or "Honay" (*Pterocarpus marsupium*).
†Weight: 56 pounds per cubic foot.



4×4 Cross Strut
S
S
Main Tie 1½"
Steel Cable
6'
T 1" Eye Bolt
Horizontal Ties
SECTION ON M M
4"×4" Cross Strut
Com
Top Bracing
1" Wire Cable
4"×4" Cross Strut
10.6"
Compression Member
6'
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T
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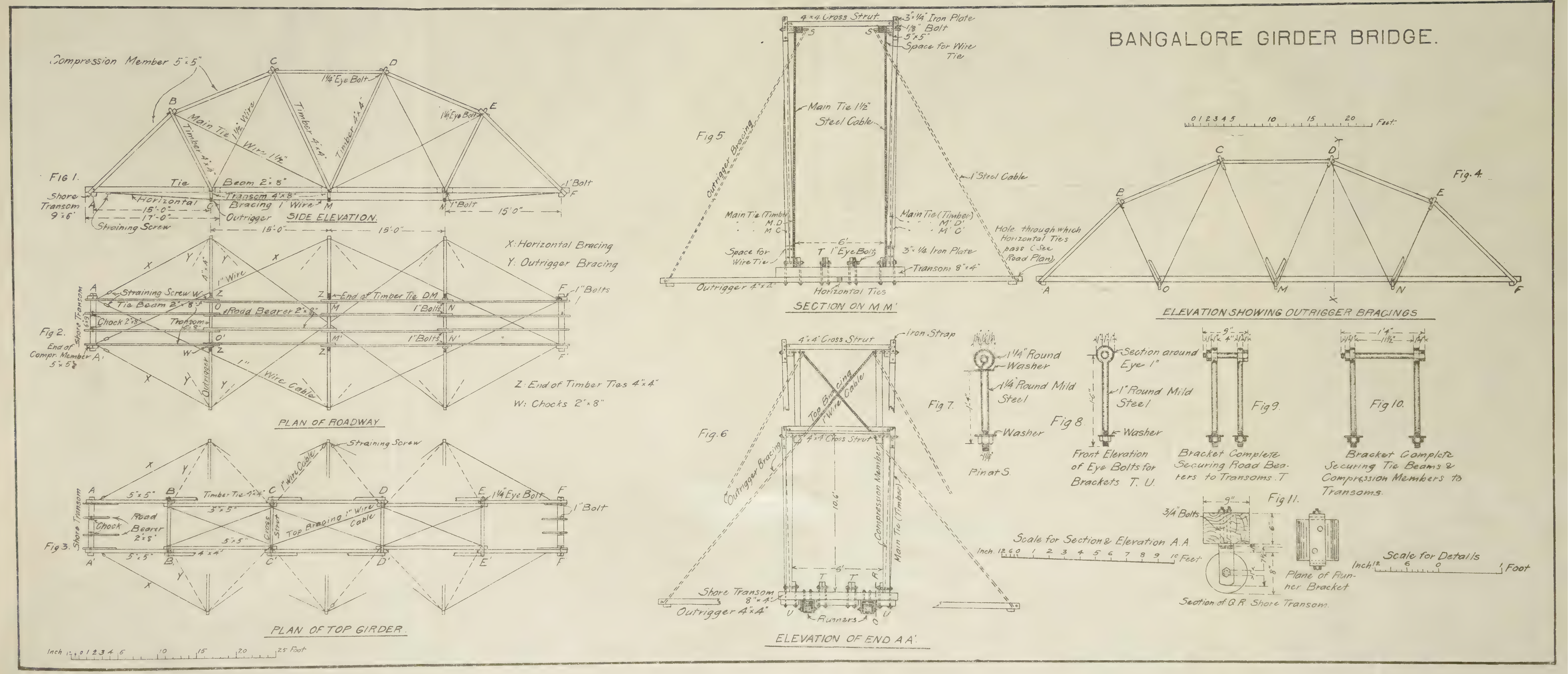
LEVATION OF END

BANGALORE GIRDER BRIDGE.



Field Girder Bridges*

BY

Maj. G. E. SMITH
C. M. G., Royal Engineers

In an article in the *R. E. Journal* of July, 1912, Major McClintock, D.S.O., describes an adaptation of the "Tarron" girder, which he calls the "Bangalore" girder. The Commandant, S.M.E., directed me to experiment with such a bridge, which I now propose to describe.

Careful readers of the *Journal* will have noticed that Major McClintock has further developed his design and published his results in the January number, and it will be noticed that the design is considerably improved. My solution differs in various respects, having been arrived at previous to the publication of the second article. My problem was to design a girder of 64-foot span to take infantry crowded in fours, and I found that by putting in a few extra road-bearers I could also deal with concentrated loads up to 75 cwt.

In the Bangalore girder it will be noticed that if loaded at O only, the girder, which is hinged at BCDEF, is unstable unless the weight of the roadway is sufficient to prevent E from rising with the thrust. Accordingly, whilst retaining three suspended transoms I used a four-sided funicular polygon and suspended each transom from all the angles of it.

Exactly what cross-bracing was put in was not clear in the July article; if wires were used they were so fine as not to be visible in the photographs. In any case, the shore frames can not have been braced, as the braces would have interfered with the headroom. The lateral stiffness appears to have depended on the pin joints. I cross-braced the frames BC, CD in an ordinary way with light planks whilst the shore frames AB and DE were cross-braced at the top and stiffened below by using rather a heavy shore transom prolonged to form an outrigger as in Fig. 3.

It should be noticed that when using very light cross-bracing it is most important to have the frames accurately squared, so that the compression members may be in vertical planes.

I found after building the bridge that the roadway swung to

*Reprinted from *Royal Engineers Journal* for June, 1913.

and fro when infantry crossed it, so I afterwards added light cross-bracing under the roadway, which made the bridge laterally very stiff.

In McClintock's bridge the thrust of the joints was taken by iron pins in wooden holes. With my heavier load the compression mem-

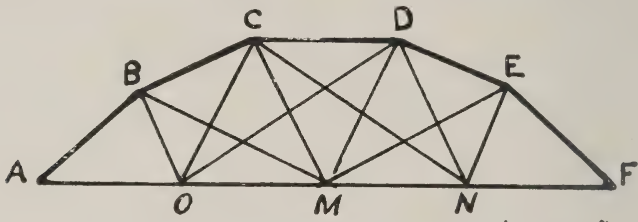


Fig 1. "Bangalore Girder"

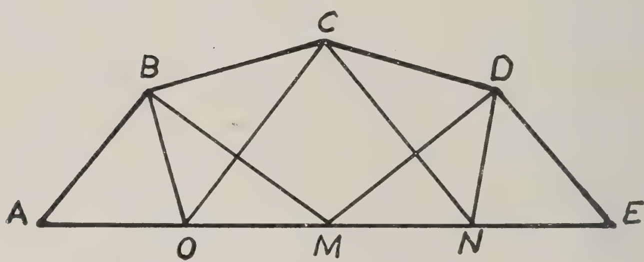


Fig 2. Tarron Girder as constructed at S.M.E.

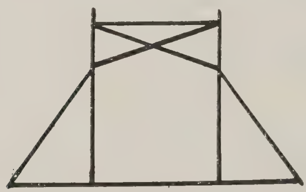


Fig 3. Shore Frames.

bers would have crushed and split, so I shod each compression member with thin iron plate to take shear off my pins.

In order to save weight I found it advisable to make all the compression members double. The narrower ones were stiffened with cover boards whilst the wider ones had cross-bracing of light boarding. Incidentally, this had the advantage of permitting the main ties to be fastened symmetrically to the center part of the pins.

I had anticipated considerable difficulty in correctly adjusting the numerous ties, but in practice I found that, by making each

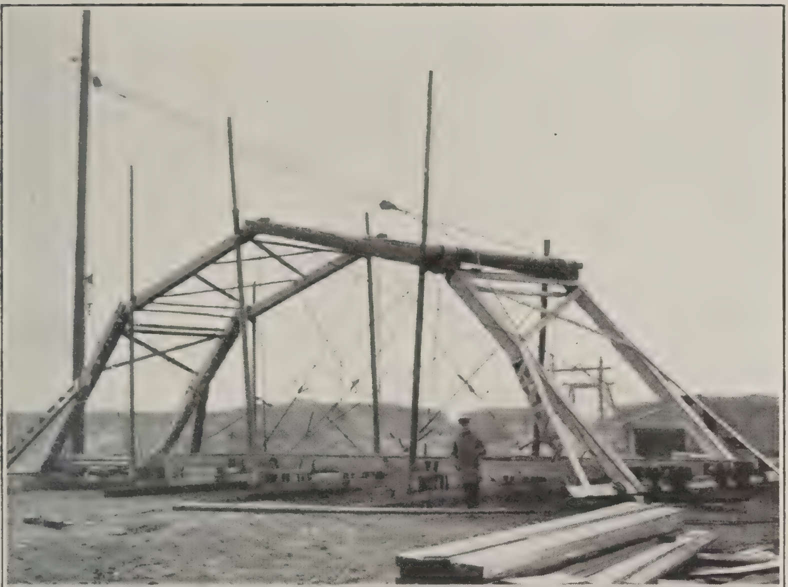


Fig. 4 (upper). View of right side of Field Girder built at S. M. E.
Fig. 5 (lower). View of left side of Field Girder built at S. M. E.

tie with two parts and adjusting by Spanish windlass, no difficulty resulted.

In erecting the girder my procedure was somewhat different. Each frame was cross-braced separately, each pin joint as soon as connected was hoisted by a pair of derricks and adjusted by them. These derricks then became suitable struts for stiffening the bridge whilst swinging. An extra spar was required to give the sling of the outer end a fair lead without bringing pressure on the light cross-bracing.

This bridge actually took 1,090 man-hours to erect, but I do not think that a field company at war strength could construct it in less than 20 hours of work. The pins are too heavy to forge on a field forge, and hoisting gear beyond that carried by a field company is required for swinging.

The weight of bridge at swinging was 6 tons, whilst the weight completed was 6 tons 16 cwt.

The bridge took a test load of 90 cwt. in a truck and showed a deflection of only $2\frac{1}{2}$ inches.

The advantages and disadvantages of this kind of bridge appear to be as follows:

ADVANTAGES.

a. It is very light and stiff for the span and load.

b. The timber used is of ordinary scantlings.

c. It can be built and swung without crossing gap to be bridged.

d. Being a complete girder, no anchorages are required after construction.

DISADVANTAGES.

a. Special ironwork has to be carried or made.

b. The stresses may be indeterminate.

c. The extra stiffening required for swinging involves extra labor and materials.

d. The frames require very accurate construction.

e. The cross-bracing can not satisfactorily be calculated, for deforming forces sideways would depend chiefly on any inaccuracy of construction and would not merely depend on wind.

f. A very heavy spar is required for the derrick for swinging.

g. The bracing of end frames can not be completely satisfactory, as headroom must be allowed.

h. For a large bridge the frames are too heavy to be readily portable.

i. Stiffness is not necessarily an advantage in field bridges, for no warning is given before failure.

In practice I should never construct a bridge of this kind unless the gap were very deep. Where 3-inch steel rope is available, a tension bridge would usually be preferable. I should only construct this type if anchorages were impracticable, and the gap very troublesome to cross. I have to thank Captain Grove for much help with this bridge.

Fig. 6 (upper). Same as Fig. 4, except bridge in place across river.
Fig. 7 (lower). End view of bridge in place across river.

Pros and Cons on the Forest and Flood Question

BY

Mr. THOMAS P. ROBERTS*
Member American Society Civil Engineers

Professor George William Hunter, A. M., is the author of "Elements of Biology," published by the American Book Company. In glancing over it I came across the following, on page 117:

THE ECONOMIC VALUE OF TREES.

Trees form a protective covering for the earth's surface. They prevent soil from being washed away and hold moisture in the ground. This they do because the evaporation of moisture through the stomata of the leaves cools the atmosphere, thus tending to precipitate the moisture in the air. Without trees many of our rivers might go dry in summers, while in the rainy season sudden floods would result. This has occurred in parts of Switzerland, France, and in Pennsylvania, where the forest covering has been removed.

This statement of Hunter's is standard doctrine, accepted as having been scientifically demonstrated. So universal is the love of trees among intelligent civilized people that it is no wonder that every statement concerning them looking to their care and increase by professors of biology, chemistry, law, theology, and physics generally is unanimously and enthusiastically endorsed, and engineers, who are only permitted to work with cold unsympathetic figures and records, have no standing whatever if they arise to interject a question occasionally.

The engineers may point to many streams in forested regions which in periods of drought dry up and, when sufficient rainfall comes, are subject to floods very much the same as streams in the open country, and ask how the "sheep" streams are to be differentiated from the "goat" streams in this respect, but in vain; for facts never militate against a spouse by his lover. They are unwelcome and will not be heard as long as the public keeps its fingers in its ears.

*Principal Assistant Engineer, U. S. Engineer Office, Pittsburgh, Pa.

The statement that "trees hold moisture in the ground," while at the same time they are known to convey much water to the atmosphere is one very difficult to maintain, being as difficult as riding two horses going in opposite directions. Engineers can never learn the trick, but it seems easy of accomplishment for men like Professor Hunter. Thus he says on page 135:

AMOUNT OF WATER LOST BY TRANSPIRATION.

A relatively large amount of water passes off by transpiration every 24 hours. A small grass plant on a summer's day evaporates more than its own weight of water. This would make nearly half a ton of water distributed to the air during 24 hours by a grass plot 25 by 100 feet, the size of the average city lot. According to Ward, an oak tree may pass off 226 times its own weight in water during the season from June to October.

In support of Hunter's statement as to transpiration of grass, Professor A. C. Church, in article "Irrigation" (9th Ed. Brit. Enc., Vol. XIII, page 362), says that a single plant of barley requires during the five months of its growth more than a gallon of water. At this rate a good crop of barley takes up nearly as much water per acre as a forest, with this difference, however, the barley must get all its water from near the surface and hence without taxing of springs as trees with their deep roots must do. Some engineers might argue from such facts that farm crops would be better than trees to conserve springs, but perhaps they misunderstand the scientists on questions of this sort and should therefore keep still.

I find that Hunter has misunderstood Prof. Ward's statement in Modern Science Series, "The Oak," edited by Sir John Lubbock, which is as follows, page 85:

It has been calculated that an oak tree may have 700,000 leaves, and that 111,225 kilogrammes of water may pass off from its surface in the five months from June to October, and that 226 times its own weight of water may pass through it in a year.

To the present writer it is evident, while Ward's language is somewhat mystifying, that it is the weight of the leaves, and not that of the entire tree, which is made the factor.

Assuming the number of kilogrammes transpired in five months as the total amount passing through the leaves in one year and dividing by 226 makes 1,085.16 pounds, weight of leaves for one tree. We then have approximately $1{,}085.16 \times 15 \times 226$ equal to 3,678,150 pounds of water which, divided by 62.5 pounds, makes 58,850 cubic feet, or about 1.35 feet, or about 16.2 inches depth

transpired from the soil covering one acre. By taking the weight of the entire tree, as per Hunter's statement, the amount of transpiration of an oak would be just about ten times that here shown, which would be absurd.

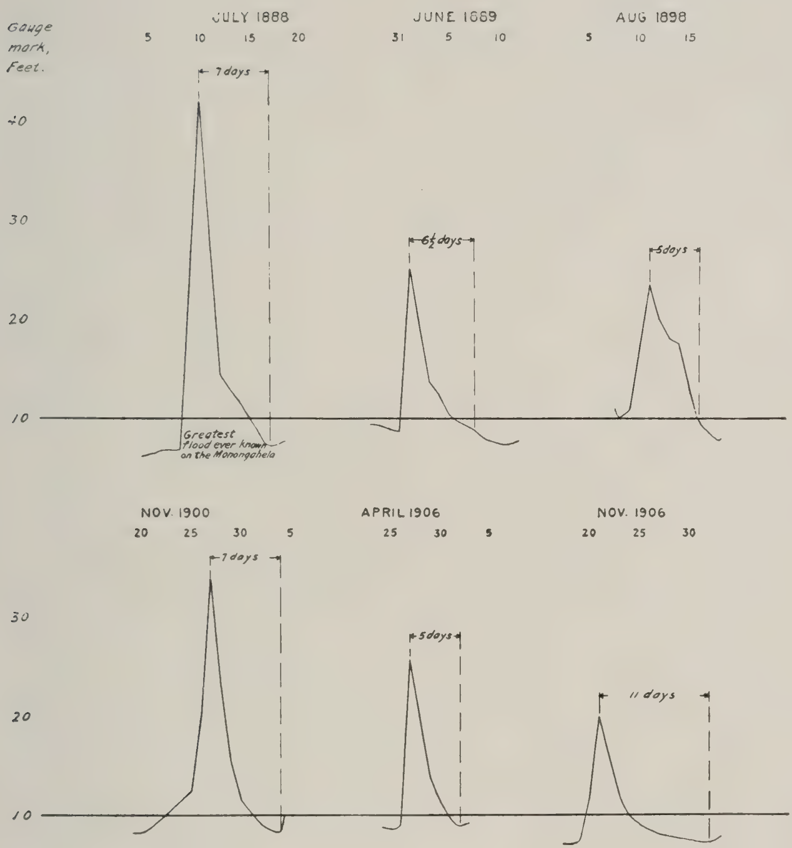
As 16 inches is nearly the entire average rainfall for the summer months referred to by Ward, we might say for his figure that it represents only transpiration *capacity*, for the trees often do well with a considerably less water supply than 16 inches during their leafy period. It is seen that Ward leaves but a scanty supply, if indeed any, to provide for surface run-off, ground storage, ordinary evaporation, and discharge of springs.

It is to be remembered that roots of trees penetrate the soil to a much greater depth than do those of ordinary field crops, which latter also are in vigorous growth only about half as long as trees. Also to be remembered that the weight of transpiring leaf surface of field crops averages very much less than that of trees covering equal areas.

Attention is called to these points on transpiration of trees and plants because its importance is not fully appreciated by the majority of engineers, who charge up deficiency in river run-off from a given rainfall simply to "evaporation." It is well for them to know that vegetation plays a very important part in such losses, and that the heavier the weight of absorbing roots and transpiring leaves the greater will be this loss. With this lesson well instilled into his system the engineer must then try to believe that trees hold moisture in the ground and that with more trees planted the more equable will be the flow of the rivers; that is, according to the biologists and botanists.

FORESTS AND RAINFALL.

It is not generally known that much of the valley of the San Francisco River, Brazil, South America, in the elevated region above the falls of Paulo Affonso, is practically a treeless desert. At some distance both to the east and west of the great river there are mountains. All about the area in question there is rainfall in superabundance with flourishing tropical vegetation. It rains copiously over the valley during four to five months consecutively; but with a change in the direction of the "trade winds" the remaining seven to eight months, for some occult reason, are too dry to support ordinary vegetable growth. Here, of all places, if there was anything in trees by their shade holding moisture in the ground, one would expect to see a natural example on an extended scale of vegetation



NOTE:

Monongahela River, Lock No 4, 41 miles above Pittsburgh. Gauge records of certain freshets following general rainstorms and showing rapidity of decline to preceding conditions, and indicating but little capacity of soil for ground storage.

conserving moisture, a region where trees could survive during water-famine months. The San Francisco valley is a case which strikingly illustrates the oft-stated fact (by meteorologists and engineers only) that forests exist only as the creatures of seasonable rainstorms, with no power of their own to increase or diminish the water supply which reaches them.*

It is the writer's observation that in periods of drought large deep rooted trees leave no moisture for saplings within their radius of action not as deeply rooted as themselves and in consequence they must die. The general notion among botanists, that the young trees which die in forests simply expire for want of sufficient light, is not, I think, of general application, as observation along streams in wooded regions, especially in the foot hills of the mountains, show extreme density of arboreal growth, as well as of subarboreal life where the small as well as the large trees and bushes have, through their roots, free access to water. The same thing is also seen in large conservatories, and in nooks of many gardens where there is no direct sunlight, where a dense foliage growth is kept up by frequent watering, but, of course, the energy required for fructification or flowering will not be forthcoming without the magic touch of sunlight.

A friend called my attention to a fact not considered though often seen in the country, brought forcibly to his attention during the drought of 1908, in western Pennsylvania; viz. the sight of corn

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*If forests could in any way induce precipitation of rain, then given time, by gradual extension, they would cover the deserts. We can find in Australia, for instance, a densely forested region (especially in the N. E. quarter) and yet tree life along the streams, which expend themselves on the interior deserts, vanishes even before the last of the water disappears. It is not as yet charged against the Bushmen that they brought about existing conditions in Australia by deforesting operations. It is not to be denied, however, for records and traditions attest to the fact that in "cycles of time," portions of the Asiatic deserts change to forested districts and again revert to deserts, in consonance with changes either in the direction of the winds, in the amount of or the periods of the rain fall. It is not necessary to assume that such changes are the result of great convulsions of nature when gentle, gradual, and almost imperceptible meteorological variations would be entirely competent to bring them about. Terrible floods, changing the course of the Yellow River no less than five times since the dawn of the Christian era, famines in Egypt, which meant protracted droughts about the heads of the Nile, have occurred at intervals of many hundreds of years and it is difficult to conceive that anything men have done has in any way been responsible for them. Man was given control of the productions of nature—plant, animal, and mineral—but as to the wind, "it bloweth where it listeth."

rows in the fields languishing for want of moisture, but where the rows extended to and beneath the shade of great "fence-row" trees, the stunted corn was *absolutely dead*.

We have here the case of corn protected from the direct rays of the sun, dying, with that exposed to its blistering heat, surviving. As the rate of evaporation from the soil is governed very largely by temperature, the conclusion is irresistible that the big trees along the fence rows were pumping water from the soil at a faster rate than were the sun's rays from the unshaded soil. It might be urged that the corn in the open was kept alive by dew which, of course, did not fall upon the corn beneath the trees, but whatever force there may be in such an argument it does not apply to the death of saplings as above referred to.

When one hears of bears and deer, after a protracted drought, seeking the abodes of man for water and recollects how far such animals could travel in the forests to find springs, the belief that forests can and do become as dry as tinder boxes, while yet cultivated fields may still be more or less flourishing, is well warranted.

As a young man in central Pennsylvania the writer helped to fight forest fires in the mountains and has seen "humus" smoldering or burning for days after the fire wave passed, and has noticed with what rapidity huckleberry bushes, for a time having a chance to get a little surface moisture, spring into existence and flourish until the later arborescent growth absorbed their water supply.

While there may be limited areas where the discharge of streams has been affected by deforestation, no impartial study of the records of discharge, high or low, of *navigable rivers* has indicated that the run-off rate per unit of time is affected by the covering of the soil, whether it be forested, cleared and left open, or cultivated.

This conclusion is really "liberal," seeing how, if Ward's statements are evenly only approximately correct, less water should flow in dry seasons from forested areas.*

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*Since this paper was prepared the writer's attention has been called to the final report of the National Waterways Commission, 1912, where under Section V, pages 28-37, is given an interesting summary of "The influence of forestation upon navigation and flood prevention." The subject is treated under the following sub-heads: "Effect of forests on stream flow," "On precipitation," "Upon run-off," "On low water flow," "Upon erosion." As a whole Section V is a very fair, but by no means complete, synopsis of a number of the important problems concerned. It contains notices of the writings of Colonels Chittenden and Burr, also of Professor Mead's report of his Wisconsin studies,

Let the disinterested inquirer consider the fact that the vast area of the Ohio valley of more than 200,000 square miles was, as found by the white man, almost wholly an unbroken forest, but is now for the greater part cleared and cultivated land. With this in mind the inquirer would naturally think that if there was anything in the notion as to the effect of forests restraining floods and increasing the low water discharge of rivers, here, of all places, the truth of such assumptions would be the most susceptible of demonstration, but to the contrary he will find nothing to support such ideas.

The inquirer will hear of great floods along the Ohio, even during the Eighteenth Century, before any settlers were permanently domiciled in the Buckeye State. Also, that considerable tributaries of the river, such as the Little Miami, were in 1785

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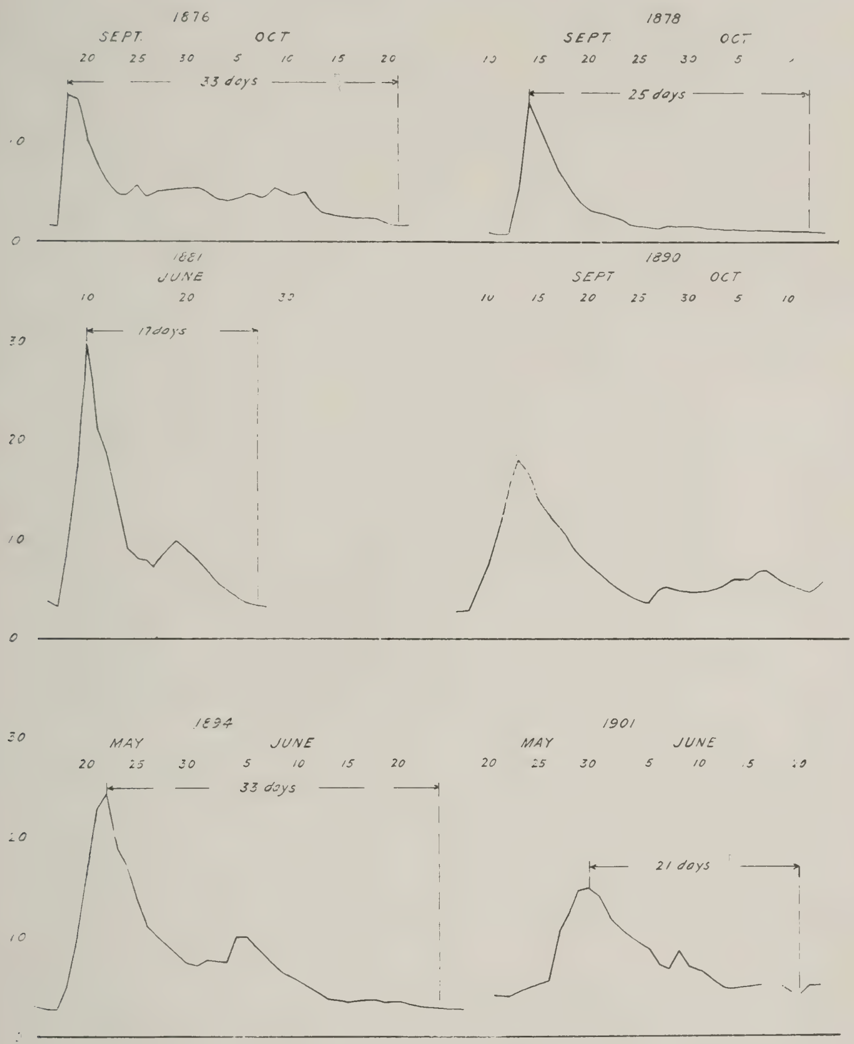
three of the best writers, who content themselves with dealing with facts and never adapt theories to imaginary conditions.

The present writer takes advantage of this occasion to say that he has long entertained the belief that attempts to show an intimate relationship between vegetation and run-off of streams leads only to negative results, and that peculiar local conditions must exist to obtain any different result.

The writer had no knowledge of Raphael Zon's elaborate paper (Appendix 5 of the Water Commission's report) while preparing his paper on "Pros and Cons." Professor Zon, in his quotations from recent foreign experimentalists, presents much information new to Americans on the subject of transpiration of trees, field crops, etc. It so happens that the present writer's statements on this subject "average" fairly well with the results quoted by Zon. The writer can not agree, however, with Zon in that part of his second conclusions (page 237) which refers to water passed off into the atmosphere by plants "bringing it again into the general circulation of water in the atmosphere and making it available for vegetation." Of course, "whatever goes up must come down," but comes down, When, and where? is the question. However, such notion is a settled dogma of the forestry faith; not heard of so much of late as formerly, but if good for trees why not for steam engines? In and about Pittsburg steam engines evaporate about 40,000,000 gallons of water daily, besides which there are the lawn sprinklers and the clothes lines, etc. Yet rain gauge records show no indication of greater rainfall in the district than in the surrounding country, and both in the city and the country about it during droughts, plants anguish as everywhere else for lack of moisture.

For Professor Zon's second general conclusion (page 273), the present writer makes decided objection to the lines which he has italicized in the following quotation. "The regularity of flow of rivers and streams throughout the year depends upon the storage capacity of the water shed *which feeds the stored water to the streams during the summer through underground seepage and by springs.*"

Objections to this tenet are embodied in what follows in the writer's remarks on Pros and Cons, and need not therefore be dwelt upon here.



NOTE.

Allegheny River, Freeport, 29 miles above Pittsburgh. Gauge records of certain freshets following general rainstorms and showing relatively slower decline than on the Monongahela River, and indicating considerable capacity for ground storage

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found to be dry at their mouths, and that light flat boats grounded at almost every rapid from Pittsburg to the Indiana State line, a distance of about 500 miles. In another year he will hear of a threatened Indian canoe attack upon Fort Pitt rendered impossible on account of low water conditions in the Allegheny River. Again, at Fort Pitt, he will hear of the river banks, including houses and gardens, being washed away by sudden floods from the Allegheny. Later, in the Nineteenth Century, in 1810 and in 1832, he will hear of floods practically as great as any since recorded, and of low water conditions in 1838 and 1854 never since equalled at Pittsburgh. As the inquirer proceeds further into the realms of official weather reports, he will learn that flood periods are exceedingly irregular. In one period of twenty-six consecutive years there were, in the Monongahela River, nearly twice as many moderate freshets as in the preceding twenty-six years, a record published far and wide by the "tree man." Yet, in this case, curiously enough, if the months of February and March, during which there are no leaves on deciduous trees, for the 52-year period, be eliminated, it will be found there were fewer freshets in the second than in the first period, which fact is skipped over by the "tree man."

The absurdity of their position on the point of increasing frequency of floods is made manifest by consulting the weather records. A "cycle" of storms on the Gulf of Mexico during the second 26-year period brought floods to West Virginia, western Pennsylvania, and Kentucky in February and March, greatest in the most forested districts of these states. Now, unless it can be demonstrated that conditions of land cover in these northern states has some controlling influence on the storms originating on the Gulf of Mexico, there is absolutely nothing in the contention of the forest people.

The inquirer will be unable to learn that people are abandoning their property along the Ohio because of the increasing frequency of destructive floods, but he will hear that with increasing valuation of their land for homes and for manufacturing purposes, inquiries are afoot as to the best methods to pursue to minimize flood damages. Among the schemes proposed is the planting of trees on some thousands of acres on the headwaters, but such a project will scarcely appeal with much force to persons of intelligence aware of the fact that if millions of acres of forests, formerly in existence,

did not avert floods, little prospect for relief would be afforded by such puny attempts at reforestation.

One thing is certain about the Ohio, viz. that whereas formerly with trees overhanging its banks they were subject to constant erosion but now its banks are more stable, the channel less changeable and the river is every way freer from snags, silt deposits, safer and better for navigation than ever before.

The least known measured discharge of the Ohio at Wheeling and Pittsburg was during the fall of 1838 and was thought to have been nearly approached in 1854 and 1856 and again in 1895, and almost equalled, as determined by measurements, in 1908. Along the Monongahela in 1838 every tributary, with one exception, between Pittsburg and Brownsville, 57 miles, became dry at its mouth. At this time the measured discharge of the river, and again in 1856, was less than has since been certainly known.

Since 1856 towns and cities embracing a population of over 500,000 have grown upon or adjacent to the river bank, which population obtains its water supply from the river. Considering this in connection with the fact that hundreds of thousands of horse-power in engines are constantly evaporating river water in the form of steam, any fair-minded hydrologist would not hesitate to assert that the extreme low water discharge of the Monongahela has increased, rather than diminished, with deforestation, cultivation of the soil, and developments of population in the valley. Many minor tributaries of the river dry up as of yore, but still the river holds its own as stated, presenting a problem for consideration.

The present writer, in 1885,* in this connection ventured the opinion that the hydrological phenomena observed on small streams, or on limited areas, if applied to large areas may lead sometimes to erroneous conclusions. It is his belief, for instance, that the dry season, or minimum discharge of considerable rivers, does not represent the tributes of a multitude of springs but rather that due to local storms, first from one, perhaps limited, locality, and later from another locality and so on *ad infinitum*, the effect of such local storms being sufficient to keep up a continuous flow in a stream which has several thousand square miles to draw upon. It is not difficult to imagine that while all of the small tributaries during the dry season would be sometimes dry, they might not all be dry together.

*See Proceedings American Forestry Congress, September, 1885, page 92.

Calling the area of the basin of the Monongahela 7,500 square miles, and its low summer discharge 300 cubic feet per second, we may entertain two theoretical propositions:

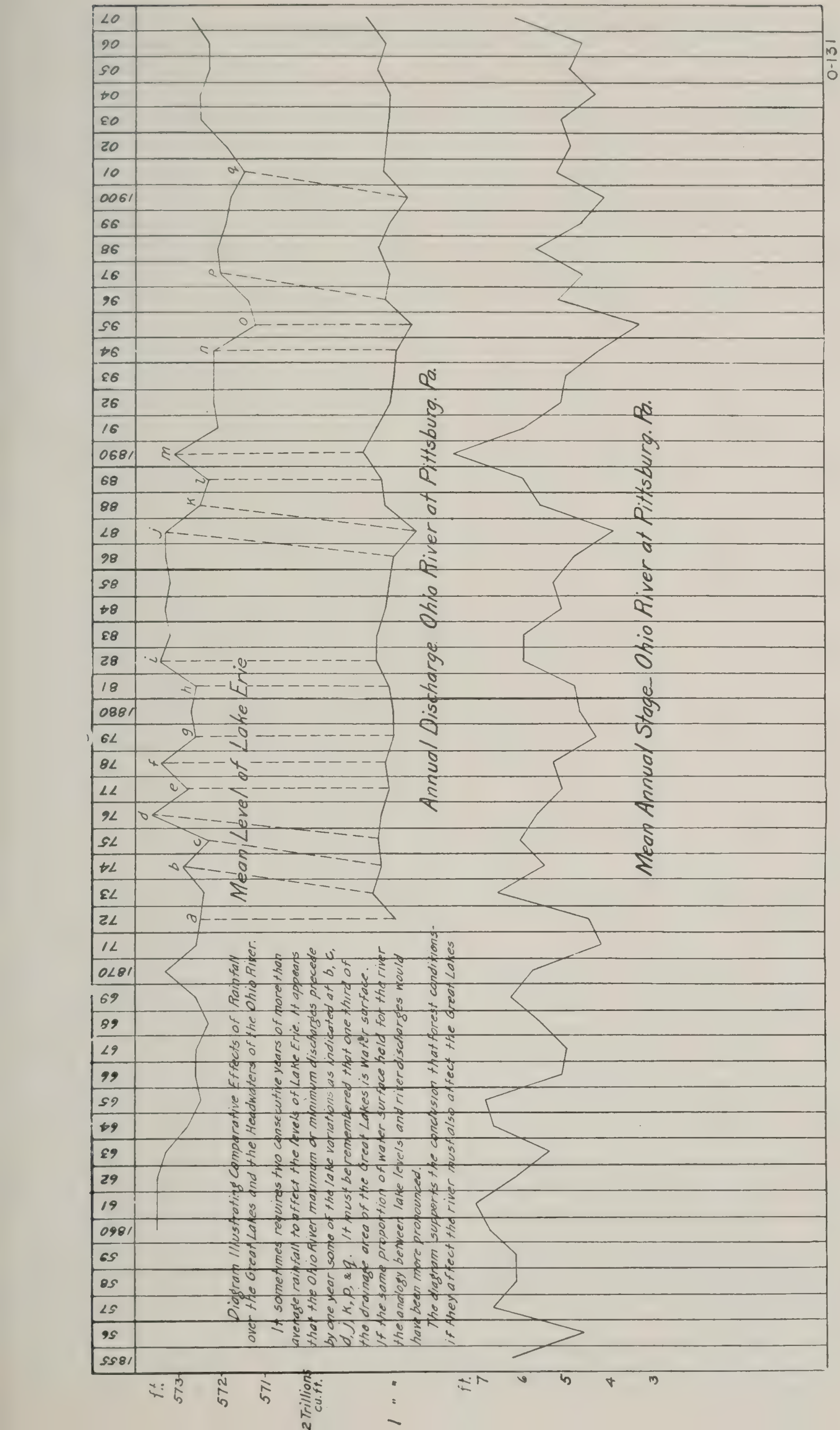
First. Sole dependence upon springs, each of two gallons per minute discharge. In this case 67,320 springs would be required, allowing nothing lost by evaporation. (As against average springs it is to be remembered that during even ordinary dry seasons they are lost by evaporation in their courses before they can unite with other springs to form brooks.)

Second. For 300 second-feet discharge dependence to be placed on local rain storms, each covering 75 square miles of territory. If from any of the hundred areas into which the entire watershed is divided there is a run-off of storm waters amounting to one-quarter of an inch depth once in 42 hours, with no allowance for evaporation, the 300 cubic feet per second would be maintained in the river.

With a little reflection it is apparent that while somewhere in the valley hard showers occurred every 42 hours, yet, at the end of the summer, it could be said on the whole the season was dry "with crops below the average." The chances of such limited storms covering rain-gage stations would be about as one for, to fifteen against, that prospect, hence few of the storms would be reported.

It is left to the reader to decide which of the propositions is most promising for effective water supply, certain to reach the river, and which accords best with meteorological and hydrological laws known to observers. The unanimous votes of the citizens of the hundred creek valleys doubtless would be cast against that of the lone river-gage recorder who at the end of the season reports, "Medium low water conditions prevailed with very uniform discharge. Guess the farmers above did not suffer for lack of rains, although around this station garden truck suffered."

In support of the above theory, tentatively put forth, it may be stated that it is rather common along the Monongahela to hear of rain storms in some of the creek valleys which raise the pools of the dams, into which the water is discharged, several inches or more while the pools above may be at a stand or falling. What the writer has said of springs, as being very insignificant sources of supply to rivers, has reference to the greater part of the basin of the Upper Ohio, where the soil is only very moderately permeable. Nothing illustrates this fact better, for instance, than the suddenness with which summer floods on the Monongahela subside. Thus,



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l
m
n
o
p
q
Mean Level of Lake Erie
Annual Discharge Ohio River at Pittsburg. Pa.
Mean Annual Stage Ohio River at Pittsburg. Pa.
Diagram Illustrating Comparative Effects of Rainfall over the Great Lakes and the Headwaters of the Ohio River.
It sometimes requires two consecutive years of more than average rainfall to affect the levels of Lake Erie. It appears that the Ohio River maximum or minimum discharges precede by one year some of the lake variations as indicated at b, c, d, j, k, p, & q. It must be remembered that one third of the drainage area of the Great Lakes is water surface. If the same proportion of water surface held for the river the analogy between lake levels and river discharges would have been more pronounced.
The diagram supports the conclusion that forest conditions - if they affect the river must also affect the Great Lakes
0-131

in July, 1888, there occurred in the valley three days of hard and persistent rains when the forests were in full leaf, yet, nothwithstanding the season, there occurred the greatest and highest general flood known in more than an hundred years' history of the valley, the river rising at Brownsville 39 feet, or to 44 feet above the river bed, a mark never before, or down to 1911, reached. The country was saturated to the limit of its capacity, yet within seven days after the crest of the flood had passed the river had fallen to almost the same low stage it had before the flood. Many such instances of rapid decline of freshets during the summer and fall months could be cited, tending to show how little capacity the country has for ground storage. The foresters often refer to the "suddenness" with which floods come up, but are wary of speaking of the rapidity with which they can decline in headwater rivers issuing from the mountains, and never do they consider the bed slopes of streams which determine the velocities, and hence the time required for a given rainfall to produce a flood at a given point. Deforestation has surely not changed the bed slopes of our rivers and their tributaries.

Quite the reverse conditions of soil may be found on some of the tributaries of the Allegheny, Beaver, Muskingum, Scioto and Wabash rivers, where the reservoir capacity of the soils in places is remarkably great. It is this great difference in soils at headwaters which gives the Allegheny River a yield of water in seasons of drought four to five times greater than that furnished by the Monongahela basin from equal areas. The Allegheny heads in a region of glacial drift, whereas on the Monongahela there are no such permeable deposits.

If indeed the entire watershed of the Mississippi was of deep glacial sand formation such as that of portions of northern Indiana and southern Michigan, known to the writer, the fluctuations in the discharge of its component rivers would be exceedingly small and there would be no necessity for improving their navigation by means of dams. The region referred to in the states named was long ago, for the most part, cleared of its timber, without appreciably affecting the conserving capacity of the soil.

Whatever effect the shade of trees may have by reduction of temperature in retarding evaporation from the soil, experiments indicate that below a depth of 30 inches in average soil there are no diurnal changes of temperature, hence whatever water or moisture may be below that depth can not ordinarily be affected by the

surface covering, although the ground must respond slowly, of course, to the mean seasonal atmospheric temperature, as does everything in the forests as well as in the open country down to 15 or 20 feet below the surface.

Some of the more enthusiastic friends of the forests who have an unshakable faith in their efficiency are now desirous of seeing what the forests can be made to do for the rivers, and they are asking how much water must be forthcoming to maintain a 6 to 9 foot depth on the Ohio without locks and dams. The magnitude of the task they propose to impose on the dear old trees is really appalling, and some of the weak-kneed brothers are already suggesting reservoirs to help the trees along. If an old father tree could speak on this topic, he would very likely say, ''We old fellows are dependent for our lives on the storm clouds; we need all the water we can get; in fact, we fail frequently to get enough to satisfy our thirst: see Hunter and Ward about this; and come to us only to unite with you in prayers to Jupiter Pluvius, for when he nods his head our leaves stick on well; when he does the reverse we are sometimes driven to take the nourishment of our offspring and see them curl up and die at our feet, and this too at times when from our lofty perch we can perceive the silvery sheen of flowing rivers far away towards the horizon. How hard and frequently it must rain down in that happy hunting ground, and yet there are those with nerve enough to tell us that it rains on these highlands more than it does way down in the valley.''

Addenda

No. 1. Condensed from ''Olden Time,'' compiled by Neville B. Craig, Publisher, 1846. Reprint, 2 oct. vols., 1876; Cincinnati, Robert Clark & Co.

Col. Richard Butler, one of the commissioners appointed to make a treaty with the Northwestern Indians, left Fort Pitt with his party of soldiers and supplies in a number of flat boats and canoes September 26, 1875. Colonel Butler kept a daily journal of the progress of his fleet, reaching Louisville, falls of the Ohio, December 7. The river was described by him as being low all the way from Fort Pitt downward, the channel in many places being very crooked; amidst sand bars and boulders their light flat boats frequently grounding. That some of the tributaries were practically dried up may be surmised from his description of the mouth of the Little Miami River, which drains 1,800 square miles, reached October 21, 1785. After mentioning that while his men were getting the flat boats off a bar nearby in the Ohio, he walked down to

take a look at the mouth of the Little Miami. Then, on page 453, he says, "Passed the mouth of the Little Miami at 3 o'clock. It was so low there was no water running above the sand bank, which is off its mouth; the sand is quick and the little water which issues through it passes through the sand."

Referring to the river channel in this vicinity, Colonel Butler remarks, "About a mile below our camp a sand bar which extends almost across the river took up three of our small boats, the others passed by by keeping close to the Indian (Ohio) shore. Major Finney remained with those that stranded."

That there had been very high floods in the river before 1785 is evident from what follows. It may be that the flood referred to was the same of which there are several independent traditions in practical agreement, to the effect that a certain flood reached a mark, where Cincinnati now stands, equivalent to a height of about 76 feet, or 5 feet higher than the greatest recorded flood, viz, that of 1884.

The Commission decided to locate their treaty station at the mouth of the Big Miami (near the Indiana and Ohio State line). They were dissuaded from adopting their first choice on the bottom land by the appearance of drift in the trees 14 feet above the level of the ground. Referring to a Captain George (probably a hunter or trapper), Colonel Butler says, on page 455, "Captain George, who had lived below the mouth of this (Miami) River, assured me that all the bank from the river for 5 miles overflow, and that he had to remove to the hill at least 5 miles back, which determined me to take the present situation." Evidently, this great flood occurred not long before 1785.

Butler's description brings to my mind personal experiences, 1866-1870, when as an assistant engineer I had charge of wing dam building and removal of snags, Pittsburg to Louisville. Our little steamer, drawing only 14 inches, during the fall of the years 1868 and 1869, frequently rubbed on the bottom at the rapids at numerous places along the river. Capt. Geo. W. Rowley, my consulting pilot, had started in as a keel boatman below Louisville, and in 1837 helped to push a boat up the river with a pole, Louisville to Pittsburg, 598 miles. The boat carried planks and stakes with which they frequently made deflecting dams to assist them in getting over the shoals. Captain Rowley was, during the Civil War, a noted pilot of a light-draft gunboat on the western rivers, and had no superior as a pilot on the first-class passenger steamers that formerly plied the Ohio.

Like all experienced river men whom the present writer has met, he invariably stated that the difficulties of navigation from low water, snags, etc., were much worse in earlier than in later times. As Mark Twain says (p. 300 in his "Life on the Mississippi"), "With these beacons, the banishment of snags, plenty of daylight in a box ready to be turned on whenever needed, and a

chart and compass to fight the fog, piloting on a good stage of water is now nearly as safe and simple as driving a stage.''

No. 3. From report of W. Milnor Roberts, Chief Engineer, dated December 24, 1839. Published in Second Annual Report Monon. Nav. Co., presented 1840:

"During the year 1838, the waters of the Monongahela, in common with most of the streams in Western Pennsylvania, were lower than at any former period within the recollection of the oldest inhabitants.

"Measurements of the stream were made at different times, in order to ascertain the minimum flow. On August 30, when the stream was generally considered near its lowest stage, the quantity passing at Brownsville was 12,420 cubic feet per minute. All the tributaries, from that point to the mouth of the Youghiogheny River, were then dry at their outlets.

"On September 19 it was again measured at Brownsville, when the quantity passing was found to be only 4,500 cubic feet per minute. This was the actual period of the lowest water that extraordinary season.''

No. 4. From annual report of Monon. Nav. Co. for the year 1854:

"In July, however, the effects of the drought began to be felt; and on the 27th of that month the water became so low that the large packet boats ceased to run during a period of nearly three months, and were not able to resume their trips until the 17th of October. During a part of that time, however, smaller boats were run; but the water was still so low that even they could carry little or no freight, and the common coal flats could not be towed over the improvement with full loads; in fact, business upon the river during that period was virtually suspended, an occurrence heretofore entirely unprecedented.

"A slight rise about the middle of October enabled the packets to resume their trips; but the water did not rise sufficiently to let out the coal boats, which were loaded for the western market, until after Christmas.

"Many of these boats were loaded in May, and kept afloat at the expense of their owners during the entire summer and fall. The water, during the greater part of that time, not only ceased to run over the dams, but by evaporation and leakage became almost literally dried out of the pools.''

No. 4. From annual report of Monon. Nav. Co. for the year 1856:

"After these repairs were completed and the navigation resumed, the river continued to fall, until the water became so low in the different pools as to suspend the navigation entirely; and *from the 14th May until the 1st December, a period of more than six months, there was at no time a sufficiency of water either in the Monongahela or Ohio rivers to float coal boats.* A rise then occurred of about 7 feet, which enabled light boats and barges to run out, but the heavily laden boats, destined for the lower Ohio

market, could not move, and still remain in the river where they are now frozen up, although many of them have been laden and prepared for market more than eight months—a state of affairs entirely unprecedented since the construction of the improvement.''

Some idea of the extreme lowness of the river may be gathered from the following statistics, which are so remarkable as almost to challenge belief; but as the measurements were made with great care, at different points and by different persons, and corroborate each other, they form strong proof of the facts.

''Below Dam No. 4 the water was gauged by the president of the company, on the 1st day of October, 1856, and the quantity passing per minute was found to be but 1,492 cubic feet. Mr. Charles Stewart, the engineer, gauged it at the Brownsville bar on the 8th day of October, 1856, and the quantity passing per minute was 1,365 cubic feet. In 1838, which was a remarkably low water season, W. Milnor Roberts, Esq., gauged the Monongahela at its lowest stage, and ascertained the quantity passing to be, per minute, 12,000 cubic feet—more than eight times as much as either of the above measurements. Charles Ellet, Esq., made examinations during the summers of 1843, 1844, and 1845 at Wheeling bar, and the minimum quantity reported by him was (September 30, 1844) about 70,000 cubic feet per minute.''

No. 5. From annual report of Monon. Nav. Co. for the year 1862.

After referring to the fact that there was a general drought in the Monongahela with very low water conditions for several consecutive months, President Moorhead quotes the following from the *Pittsburg Gazette*, which he says is ''mainly correct.''

''The river continued to swell steadily until about noon yesterday with 11 feet by the marks. As usual, this rise is almost entirely out of the Allegheny, as the Monongahela has not risen a foot all told. This makes the third or fourth rise we have had in the former stream this fall, while the latter has not risen 2 feet during the past ten months. This is certainly very strange, yet it is nevertheless true.''

Marked seasonal differences in the rainfall covering the two valleys has also been observed in more recent times.

Referring to the drought of 1879, President Moorhead says that on May 1 the Ohio at Pittsburg had fallen to 4 feet 6 inches, and between that date and November 13, 6½ months, there was at no time a sufficiency of water in the Ohio to float coal barges, with the exception of one day when there was 7 feet 6 inches depth, and that during the entire time there was but six days when it exceeded 4.6 feet at the city marks.

This statement regarding 1879 is about equalled in Neville B. Craig's ''History of Pittsburg,'' page 285, where Major Craig, Quartermaster General, speaking of the building of the *President Adams* and the *Senator Ross*, small war galleys, mentions that

the *Adams* was launched May 19, 1798. In letter dated July 27, 1798, Major Craig says "The galley *Senator Ross* is ready to launch but not enough water to float her. The river is said to be lower than ever known at this season." October 13 he writes "The rivers still continue to fall and are now said to be 12 inches lower than ever before known." Neville B. Craig adds "The river continued low until closed by ice, so that the galley, *Senator Ross,* was not launched until the spring of 1789."

Speaking generally of the Ohio, Craig, the historian, quotes a letter from Major Craig to General Knox, Secretary of War, dated June 14, 1793, and says, "The navigation of the Ohio is not materially better from Wheeling than from Pittsburg in a dry season, and our best Ohio pilots say they find nearly the same difficulty until they pass the rapids below the Little Kanawha; indeed, we have found that more accidents have happened to boats and more loss sustained below Wheeling than above." A statement which he adds is "Confirmed by letters from Col. John Gibson and Maj. George McCully, both men acquainted with the river and long accustomed to navigate it."

Pivots in Defense: Their Size and Organization*

By Dolf

Field Service Regulations, Part I, lays down (Sec. 107. 9) that "defensive positions will normally include a number of localities of special tactical importance," and that "the efforts of the defender will be directed in the first instance to occupying and securing these points." Here, then, is a definite principle on which the defense of a position or zone should be conducted. According to the Manual of Field Engineering (Sec. 50.3), these localities may be commanding features of the ground, groups of substantial buildings and enclosures, or wooded knolls. It has been pointed out (*vide* "Natural *Points d'Appui,*" *R. E. Journal*, February, 1913) that localities such as woods and villages, which are so to speak ready to hand, are better than those which have to be constructed for the purpose, such as a group of trenches on a hill. The following reasons were given; woods and villages are easily adapted for defense; abundant material for obstacles, etc., is usually available; they afford cover from the view and fire of the enemy and serve to screen the movements of reserves; and, perhaps most important of all, they give almost complete concealment from hostile aerial reconnaissance.

It can not be expected, however, that natural *points d'appui* or pivots (to use the official nomenclature) of suitable size and situation will always be forthcoming. It seems clear then, that if this system is to be followed "natural" pivots may have to be enlarged, or entirely "artificial" ones constructed.

Thus the question arises as to what is the suitable size for a pivot? for if one has to be made to order, some idea as to this point is certainly necessary. Before going into this it may be well to recapitulate the objects and advantages of this system of defense. The objects may be said to be

- i. To regain to a certain extent initiative presumably lost owing to defensive attitude; this is accomplished by occupying localities that are certain to attract the enemy to attack them.
- ii. To preserve an offensive spirit in the defense by giving opportunities for counter-attack.
- iii. To economize the numbers required for defense by occupying only part of the front.

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*Reprinted from *Royal Engineers Journal* for June, 1913.

iv. To facilitate command and organization of the defense by the concentration of the defenders into nuclei.

v. To obtain full benefit from the power of modern firearms and the disconcerting effect on the enemy of flanking and cross-fire.

Other advantages not included in the above may be said to be

i. The gain in *morale* due to an increased sense of security which results from the shape of the pivot, its internal organization, and the facilities for strengthening it.

ii. The mutual support of neighboring pivots.

iii. The possibility of developing flanking fire from positions screened from the front.

iv. Ground can be used to the best advantage.

Almost the only disadvantage is the liability to concentrated hostile artillery fire. This can be lessened by avoiding overcrowding and making good cover.

To attain the objects and secure the advantages enumerated above, a pivot must be of sufficient size and suitable shape to give the necessary amount of fire to front and flanks; it should require for garrison a complete unit or an easily handled portion of one; it should be large enough to be comparatively invulnerable to artillery fire. These are not very definite data from which to fix the size of a pivot, but it is necessary to make the best of them.

The length of front required depends upon the amount of ground to be covered by fire. This is determined by the extent of field of fire to the front, and partially by the interval between neighboring pivots; both these depend again on the nature of the ground both as regards shape and surface in so far as visibility is concerned. The amount of fire to a flank which is required also depends upon the extent of field of fire in that direction. If the ground is open, it would seem desirable that the whole space intervening between two pivots should be under the "close" rifle fire of one or other; this would give 1,200 yards as a maximum interval; either pivot would then be able to sweep the front of the other with "effective" fire. Under these conditions each pivot would be responsible for 1,300 yards or so of front. The maximum garrison excluding local reserves for this frontage, calculated according to Sec. 108. 8, *Field Service Regulations*, I., would be 1,950 men, or two battalions.

The conditions assumed are, however, very favorable to defense; the ground is open and there are facilities for forming strong pivots; the method, as such, aims at holding certain portions of the line only, so that a larger proportion of the troops may be available for counter-attack. It seems therefore that a battalion would suffice for the garrison of a pivot in the conditions described. If the ground is such that the field of fire to front and flanks is more limited, the interval between pivots may have to be reduced. On the other hand, the enemy will be under fire for a shorter time and more rifles may be necessary to produce the required effect. Within certain limits these factors would neutralize each other and a

battalion would remain theoretically the most suitable garrison. It is obvious that, when the field of fire to the flanks is very limited, smaller garrisons will suffice.

It may be said that such theoretical discussion has little practical value and that each case can only be dealt with on its merits. It is certain that each case must be dealt with on its merits, but previous theoretical discussion should facilitate this being done quickly and correctly.

In the defense of a position on this system, therefore, an estimate should first be made of the average garrison for a pivot demanded by the type of country. For ordinary European country, a battalion to a pivot seems a good basis to work on, if the necessity for modifications imposed by abnormal conditions is kept in mind. This, of course, applies to operations of the type contemplated in Secs. 107 and 108, Ch. VII, *Field Service Regulations*, Part I—that is, stubborn defense by a considerable force.

Other conditions, as in the case of a protective detachment, may require a smaller force to hold a more extended line; the average pivot garrison would then be proportionately reduced.

''Natural'' localities, or possible sites for artificial pivots, must next be examined. If the former are unsuitable as regards size or in other respects, it is for decision whether the advantage of concealment from air craft should outweigh other considerations, such as undue concentration and consequent liability to suffer from artillery fire; whether, in fact, the locality should be occupied as it stands, enlarged ''artificially,'' or a fresh artificial pivot constructed on a more suitable site.

It may happen for strategical, or even tactical reasons, that there is no objection to the enemy discovering the position and that there will be no advantage in concealment from aircraft.

Concealment from the view of hostile troops on the ground is a different matter and should always be arranged for as far as possible.

In applying this system of defense to the ground there is a tendency to hold minor features or localities. This should be guarded against, as it leads to the formation of a line of small posts. Such a line has few advantages over the old linear method and many disadvantages, especially as regards organization, compared with the system of large pivots.

There seems to be a certain amount of vagueness about the terms ''pivot'' and ''post.'' Neither is defined in any training manual, except the former in its cavalry signification.

Every defended locality is not likely to be a pivot in the original meaning of the word, but the term seems to be the official one for what the French call *points d'appui*, or what Major Swinton in his lecture at the R. A. Institution on February 16, 1911, characterized aptly as ''defensive blobs.'' The customary use of the word ''post'' implies something on a smaller scale, and it would perhaps be well if the distinction as to size were generally recog-

nized. One could then say without fear of misunderstanding that, if the shape of the ground or tactical conditions demanded that a "pivot" should cover an area somewhat extended for its garrison, it could be occupied by holding "posts" at intervals connected by obstacles.

One of the advantages claimed for the pivot system was the improved facilities for the command and organization of the defense. This is due of course to the concentration of the defenders into well-defined localities, instead of their being distributed along the whole front. Communication (in both senses of the word) to the rear is much simplified.

The internal organization of the pivot must also be good, if full value is to be derived from the system.

The garrison of a pivot will usually consist of the firing line and supports, local reserves being kept outside for use in intervals, etc. No great measure of offensive action will therefore be expected from the pivot itself; and the proportion of the garrison in the firing line can for this reason be relatively high. Men so allotted to the front line beforehand will be of more use there than if hurried into position at the last moment; and, if suitable internal communications have been made, they can always be moved to reinforce a more threatened portion of the defenses.

Screened communications are very important; these must be both lateral, along the front line, and radial, back to the center, where headquarters, reserves, etc., will usually be located. The enemy should not be able to detect any movements in, or behind, the firing line. If time is available such communications should be made bullet proof.

The shape in plan of a pivot generally gives a fair indication of its suitability; if there is a great disproportion between its breadth and depth, there should at least be some good reason for it.

It is better, if tactical considerations permit, not to allot to a pivot a garrison which does not admit of a sound system of command and organization. A complete unit, a company or battalion, whose commander is known to all is obviously the best; two companies or two battalions are probably the worst, as the command of, say, a battalion by the colonel of another in addition to his own can never be a really satisfactory arrangement.

Within the pivot, the subdivision between units should correspond with tactical requirements; better results may be expected if, for instance, companies do not unnecessarily have to face both to front and flank, but can be given a definite tactical mission such as sweeping with fire the front of a neighboring pivot.

A so-called "Keep" may be prepared to enable the garrison to continue to resist after they have become too weak to hold the original outer line, or so that, owing to changed tactical conditions, the pivot can be held with a reduced garrison. Each case must be considered on its merits; it will often be undesirable, and sometimes impossible, to make a "keep."

It is laid down in the Manual of Field Engineering (50.3) that "pivots should be capable of all-round defense." It would seem that this should be interpreted to mean that the flanks should be brought well back, so that the defense would not be seriously affected by even the loss of a neighboring pivot.

The inadvisability of doing anything which would assist the enemy to hold the pivot if he captured it, and the extra work which would be entailed by making defenses actually facing to the rear, would as a rule counterbalance the advantage of being prepared for the somewhat unlikely contingency of an attack from that direction.

The *Plate* shows suggested organization for two such pivots in fairly typical country. It may serve to illustrate most of the points mentioned above. No. 1 is supposed to be the right flank pivot of a position occupied by a brigade. Both are liable to be exposed to artillery fire. Their extent is, however, such that they would not be greatly affected by it. No. 1 can develop plenty of fire to the exposed flank. It is perhaps a little weak in frontal fire; wing trenches A and B would be used to increase this if necessary.

The distance apart is in this case decided by the shape of the ground; the *central* interval works out at about 1,200 yards. A battalion would be a suitable garrison for each pivot, assuming of course that a stubborn defense is intended. This may appear excessive in the case of No. 2, but owing to the contour of the ground the defense of its left flank could not be assisted from the next pivot. In both cases "natural" cover is available; the hedges round the village are thick and high, and "East Wood" is a well-grown hazel copse with some large trees. Any clearings required would not be conspicuous, as they are a common feature of such woods. There are good existing radial communications in No. 1, and the lane along the front of East Wood would serve for lateral communication in No. 2. Others would have to be made. The shape of No. 1 may be said to be suitable; No. 2 is somewhat broad for its depth, but this is dictated by the shape of the ground.

If desired, a "keep" could be arranged for in No. 1 Pivot in the area, "Parsonage Farm, St. Mary's Church School." It would be practically impossible to form one in No. 2. An entrenchment could of course be made in rear, say, in the neighborhood of C, but it would have little connection tactically with the primary defense of the pivot.

As regards the occupation of such positions, the approximate sites of pivots would be given and the garrisons allotted by the brigade commander, working on results of reconnaissance carried out by or for him or by divisional or higher authority. The pivot commander would then be responsible for the detailed arrangements for which any report or sketch made by a reconnoitering officer would be most useful. To assist in the work some Sappers would no doubt be allotted if available. As we have only two-thirds of a Field Com-


PIVOTS IN DEFENCE.
Scale
North
East Wood
3 Cos.
C
1 Co.
M.G.
No. 2 Pivot.
Church Form
St Marys Ch
School
Manor Farm
New Farm
2 Cos.
B
A
No. 1 Pivot.
550
500
Fire trenches, etc.
Communication do:
Clearing
Abatis

pany to an Infantry Brigade, one section is all that can be expected for a pivot occupied by a battalion.

The R. E. officer should be the pivot commander's adviser as to the employment of the Sappers, and in all other technical points connected with the defense. These are outside the scope of this article, which has dealt with purely tactical considerations; technical details are, however, so closely allied to what may be called tactics of ground that there seems no doubt that all R. E. officers should be experts in the latter subject as well as in the former, especially as it is laid down in Engineer Training that the reconnaissance of the ground and proposals for its occupation may fall to the lot of Engineer officers.

Some Experiments in the Use of Bamboo for Hasty Bridge Construction

BY

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In pioneer work with the advance of an Army the Military Engineer is called upon to make the best use possible of such materials as he can find at hand. There are localities in the East where bamboo is the only material available. Such a locality is the north central part of the island of Luzon, where one may travel miles without seeing any other vegetation worthy the name of material for construction. Bamboo is not in general as suitable as timber for the construction of military bridges, yet it possesses some advantages, as will be noted.

Some interesting examples of the use of this material may be seen in the work of the Filipinos, who employ it extensively for a great variety of purposes. It is even used for water pipes, pumps, bellows, and other intricate devices, and a house is never built without it. The native bamboo bridges are in general built in a rather slip-shod fashion, on no sort of system; and hardly any two bridges are alike. In these bridges bamboo is generally used in conjunction with other materials. No native bridge that the writer has seen is suitable for heavy Army traffic, for the reason, principally, that as the natives use no heavy traffic they have no occasion to construct bridges of great strength. The writer undertook to devise some forms suitable for military purposes, and the results of his experiments are given herein.

PHYSICAL CHARACTERISTICS OF BAMBOO.

Bamboo grows extensively in the Philippine Islands. It occurs in clumps of from ten to thirty stalks from a common root. The trees are of rapid growth, and often attain a height of 60 feet and a diameter as great as 7 inches at the butt. There is, ordinarily,

little difficulty in obtaining fairly straight pieces of nearly uniform cross section, 30 feet in length and 4 to 5 inches in diameter. The structure is too well known to require description. The joints occur at intervals varying from a few inches up to 2 feet or more. The cross section between joints is an annulus, the thickness of the annular ring as well as the diameter of the stalk decreasing toward the top of the tree. At the butt the tree is sometimes almost solid, whereas at the tip the thickness of the fiber may be not more than 1/8 of an inch.

The branches from the main stalk are small, and the fiber is very straight and, except for the joints, free from knots. Bamboo has little strength across the grain. It splits very easily, and as a consequence the stalks crush easily. The tensile strength is very high. The compressive strength is considerably less, due to the fact that failure under compression results from the splitting of the fiber as well as actual crushing. Columns of a length of more than ten diameters fail by splitting and brooming of the fiber near the ends in cases where a wooden column of the same dimensions would first buckle near the center. The strength of columns of moderate length can therefore be increased by binding the ends with wire or marline. There are several different species of bamboo, showing considerable variation in strength. The best grade is known as Espina No. 1. It can be distinguished by having finer thorns than the inferior varieties.

Bamboo is subject to rapid decomposition if alternately wet and dry. It is also quickly attacked by ants.

The advantages of bamboo for the pioneer's use are as follows: It is very light in proportion to its size and strength, and easily cut and handled. Its annular cross section adds greatly to its strength, both columnar and transverse. It occurs in sizes convenient for use.

The disadvantages of bamboo are as follows:

It splits and crushes very easily, and can not be secured by either spikes or bolts. (Seasoned bamboo, before it begins to rot, has in general more strength than the green material.) Lashings will not hold securely if any great strain is brought upon them. Bamboo cannot be notched or dapped without seriously diminishing its strength. It is difficult to cut down and drag out of the clumps in which it grows, and cannot be obtained in large sizes.

The results of certain strength tests are given at the end of the article.

TYPES OF TRESTLES.*

The ordinary framed trestle bridge will be the best form in the great majority of cases. Other forms of bridge should be used only where it is manifest that they are more suitable than the framed trestle, which will seldom be the case. In a very narrow and very deep ravine a spar bridge will occasionally be more suitable than a trestle bridge (see Fig. 5), although even in such cases a tall trestle in the middle, or two of less height near the banks, will often be easier to install than a spar bridge.

In constructing trestles of bamboo, some restrictions are naturally imposed. Lashings only can be used for fastenings, but these may be supplemented by pins of hard wood or bamboo. Lashings will not hold under any considerable strain, and a plain butt joint is the only safe kind. As bamboo can not be readily obtained in large sizes, additional strength must be secured by the use of multiple pieces.

The various trestles shown in the drawings and photographs were constructed and tested by Company H, Second Battalion of Engineers.

Fig. 1. This trestle is very simple and easily constructed. The cap and sill are smaller than the legs, and secured in notches cut in the latter, and resting directly upon the ends of the legs. There are four diagonal braces, which may be made of round sticks or split bamboo. The whole is fastened together with pins and lashings. Considerable stiffness may be added by extending the diagonal braces and attaching them to the ends of the cap and sill (see Fig. 3). There may be any number of legs, and the outer ones may be slightly inclined. This trestle is suitable for light traffic, such as infantry, cavalry, and pack trains, and for moderate heights up to about 10 feet.

Fig. 2. This trestle is quite similar to the last, except that the cap and sill are larger, and are secured to the legs by splices of bamboo taken from near the butt of a stalk. It is much stronger than the last type, but takes longer to construct. If the height is over 8 feet, two horizontal braces should be placed across the legs at the middle. These braces may be let in between the

*The first nine figures, of which only six are referred to in the text, will be found on the lithograph between pages 596 and 597.

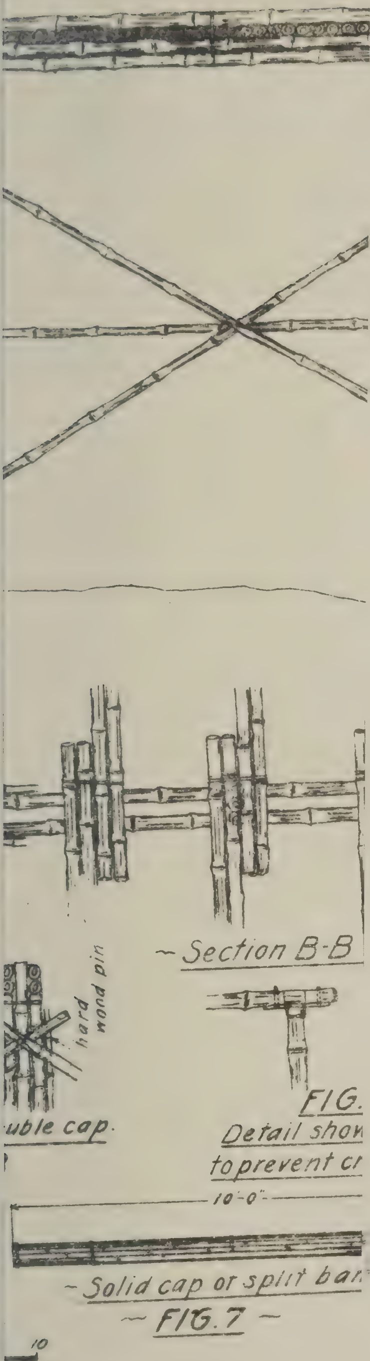
legs or lashed continuously clear across (see Fig. 6). This trestle is suitable for bridge trains and loaded escort wagons, or any lighter traffic.

Fig. 3. In this trestle the cap, sill, and legs are each made of four pieces (stalks) of bamboo lashed together at intervals. Its construction is very simple; it does not require straight pieces, and the legs afford large bearing surfaces for cap and sill. It is fastened and braced in a manner similar to the other types. This trestle is suitable for the heaviest traffic of an army, and for any height up to 25 feet.

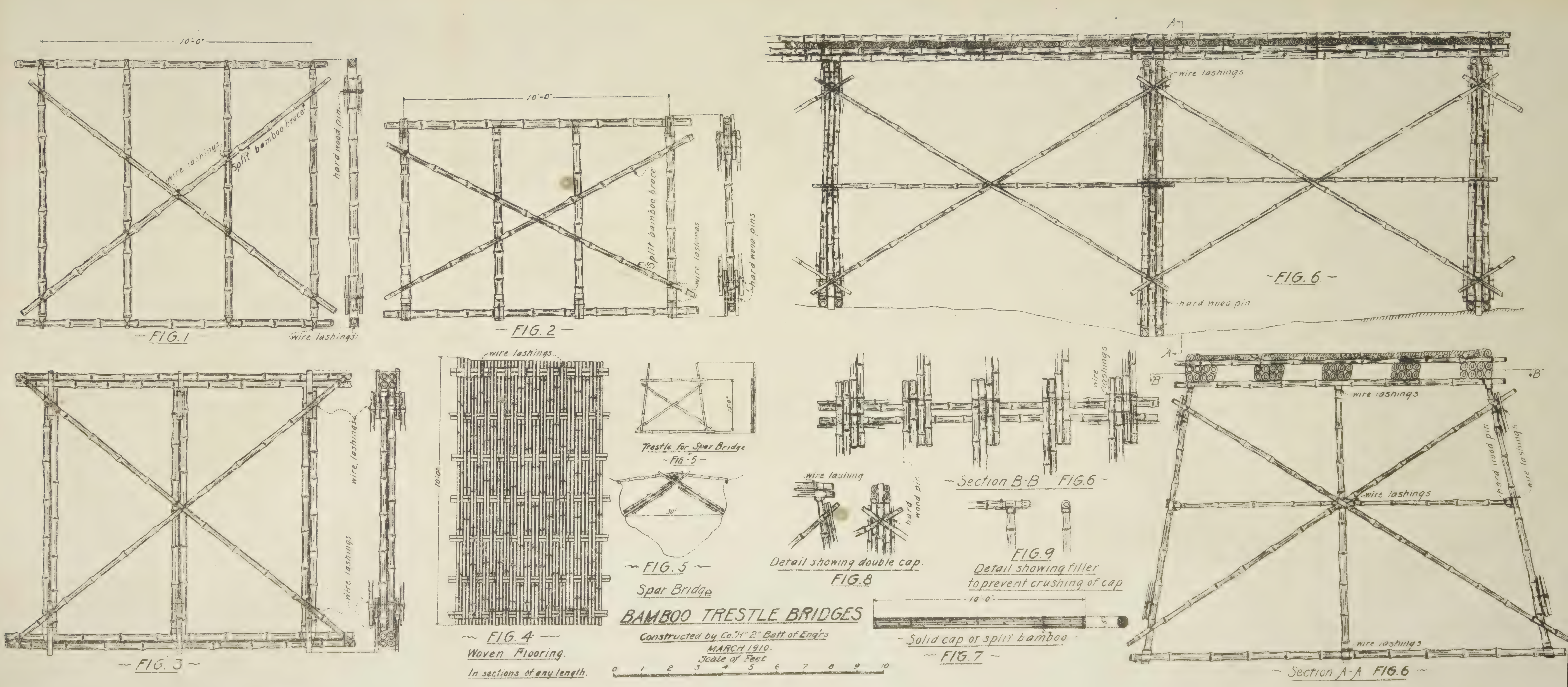
Fig. 6. This trestle has three legs of three pieces each. The middle piece of each leg supports no weight except that transmitted by the lashings, but serves to support the leg against buckling, and forms an attachment for the cap and sill. The latter are each double, and rest upon the two outer pieces of each leg, being lashed securely to the middle piece of the leg. Both cap and sill may be made quadruple (see Fig. 8). The trestle is diagonally braced, and in heights over 10 feet the legs should be horizontally braced at the middle. The two outer legs are slightly inclined. The center pieces of these legs may be extended upward to form hand-rail supports. This trestle is suitable for heavy loads and high lifts, up to 25 feet. For low lifts and light traffic it is not as suitable as Figs. 1 and 2, since it requires more material and more time to construct. This excellent type was devised by First Sergt. Geo. S. Martin, Company H, Second Battalion of Engineers, at the writer's suggestion.

Fig. 5. This trestle is a modification of the last, designed for use in a spar bridge. Its construction is shown by the illustration. It may have two or more legs, each composed of two pieces. The lower angle between the spars when set in the bridge should be greater than 90 degrees, because if it is less there is a tendency to push the transom piece off the legs which support it. The roadway bearer is a bundle of several stalks of bamboo. A bridge of this type is suitable for light traffic only. As already mentioned, it should never be used where an ordinary trestle bridge is practicable.

The trestles heretofore described are utilized in the same manner as equivalent timber trestles. The balk are made of bundles of from three to five bamboos, four pieces being generally the best. They are securely lashed together at the ends and at several intermediate points, to make them act to some extent together, like the faggots in


~ Section B-B
hard wood pin
uble cap.
FIG.
Detail show
to prevent cr
10'-0"
~ Solid cap or split ban
10

~ FIG.7 ~



10'-0"
wire lashings
Split bamboo brace
hard wood pin.
wire lashings
- FIG. 1
10'-0"
Split bamboo brace
wire lashings
hard wood pins
~ FIG. 2 ~
wire lashings
hard wood pin
- FIG. 6. -
wire, lashings
wire lashings
~ FIG. 3 ~
wire lashings
10'-0"
~ FIG. 4 ~
Woven Flooring.
In sections of any length.
15'-0"
Trestle for Spar Bridge
- FIG. 5 -
30'
~ FIG. 5 ~
Spar Bridge
wire lashings
~ Section B-B FIG. 6 ~
wire lashing
hard wood pin
Detail showing double cap.
FIG. 8
FIG. 9
Detail showing filler to prevent crushing of cap
A
B
B'
wire lashings
hard wood pin
wire lashings
wire lashings
wire lashings
~ Section A-A FIG. 6 ~
BAMBOO TRESTLE BRIDGES
Constructed by Co. "H" 2nd Batt. of Eng'rs
MARCH 1910.
Scale of Feet
0 1 2 3 4 5 6 7 8 9 10
10'-0"
- Solid cap or split bamboo -
~ FIG. 7 ~

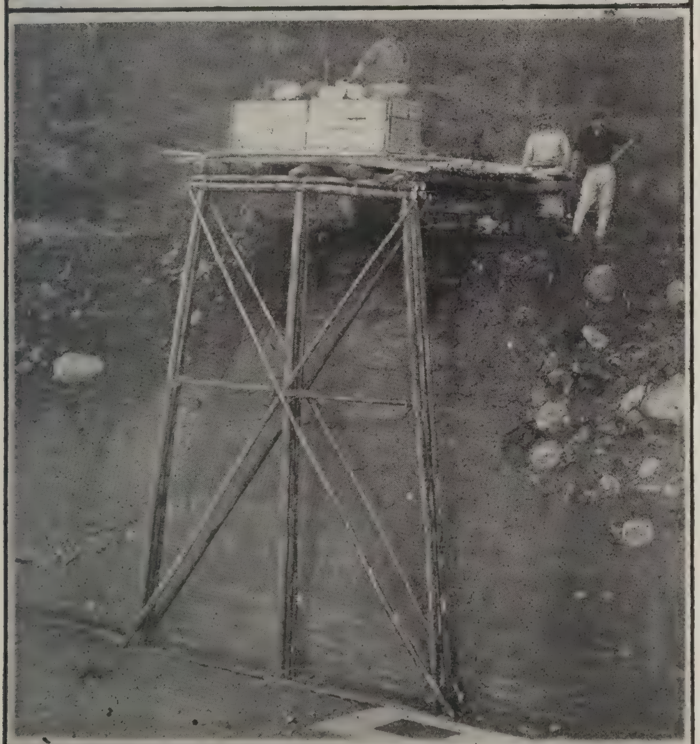


Fig. 10 (upper). Same trestle as described on page 596 and illustrated in Fig. 5 of lithograph. Fig. 11 (lower). Nearly same as Fig. 10, except there are only three pieces of bamboo in each trestle leg.

the fable. The number of balk depends on the span. Ordinarily five are sufficient for loaded escort wagons. The roadway is made of bamboo poles, brush and earth, in the usual manner (Fig. 6) or of the wattling described hereinafter. There should be longitudinal bracing between trestles in each bent to prevent racking. The greater the height of the trestles the more elaborate should be this bracing (Fig. 6).

As to the length of span, the usual economic principles apply. A span of 12 to 15 feet was adopted in these experiments. As balk are easier to construct than trestles it may at times be economical to decrease the number of trestles by increasing the span, not to exceed 20 feet, using larger balk or more of them.

The wattling shown in Fig. 4 was constructed for use as flooring. It is made by splitting the stalks of bamboo into two or three pieces and weaving together, as shown. The pieces running across the roadway are placed close together, the longitudinal binders farther apart, and the whole secured with wire. The convex side of the fiber is laid upwards. This floor can be woven very quickly, and is quite suitable for light traffic. If doubled, or laid on six balk (on a 10-foot roadway) it will carry loaded wagons. If subjected to heavy traffic for any considerable time it should be covered with grass and earth. This wattling, being very light, can be easily carried from place to place in wagons, and if prepared in advance its use considerably reduces the time required to construct a bridge. A wattling of this form is also very useful as a revetment.

TESTS OF TRESTLES AND BALK.

Two trestles of the types shown in Figs. 1 and 2 were tested up to 5,000 pounds each. One of them endured the test without any signs of failure. In the other the cap, being made of a weak piece, crushed and collapsed. The failure of the cap was so gradual that the men forming the test load had time to leave the bridge. A trestle on the design shown in Fig. 6 was tested up to 6,000 pounds, which was all that could conveniently be put upon it. Under this load it showed no signs of weakness. The height of this trestle was 21 feet. Two balk, consisting of three stalks each, were tested on a span of 20 feet. They failed under a load of 1,200 pounds (600 pounds each). The average diameter of all the stalks was 2.85 inches and the average thickness of the fiber, 3/8 inch. Two balk, consisting of four stalks each, were tested on a span of 12 feet. They failed under a load of 5,250 pounds. The average diameter of the sticks was 3.375

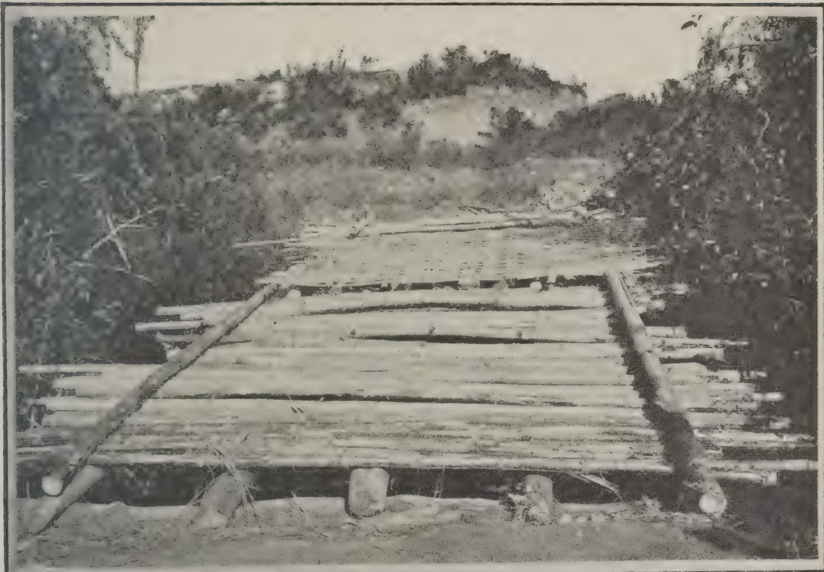


Fig. 12 (upper). Showing flooring system, using both ordinary bamboo poles and wattling made of bamboo. Both to be covered with earth, straw, etc.
Fig. 13 (lower). Shows simple bamboo trestle and balk made of 4 bamboos each.

inches and the average thickness of the fiber 1/4 inch. Of the test loads referred to, the first (1,200 pounds) was concentrated at the center of the two balk, whereas the other load (5,250 pounds) was uniformly distributed. The wattling was tested by being covered with men placed as closely together as possible, which test it endured without damage.

REMARKS.

Lashings may be of rope, vegetable fiber, wire, or other suitable material which may be available. Wire is the best, but great care must be exercised to insure tight lashings, and frapping turns should be used wherever possible. Pins should be set snug, but not tight, and if a pin is used near the end of a piece there should be a turn or two of wire between the pin and the end to prevent splitting. If the upper end of the leg be cut so as to fall exactly at a joint, a better bearing for the cap is afforded. The diaphragms at each joint divide the bamboo into a number of water-tight compartments. If the trestles are placed in water these diaphragms should be punched out with an iron rod, or a hole cut into each compartment. Otherwise the buoyancy of the trestle will make it difficult to place.

The trestles shown in Figs. 3 and 6 can, with reasonable care, be made strong enough to safely carry the heaviest traffic. The principal weakness developed by the lighter trestles (Figs. 1 and 2) was the crushing of the single caps or sills. The crushing of the sill is not apt to be serious. The crushing of the cap may be prevented in a number of ways, as follows:

a. By using a timber scantling set on edge, or a piece of round hardwood for the cap. The length of the cap need never be more than 10 feet, and timber of this length can often be found when the greater lengths required for the trestle and balk might not be available.

b. By using five legs in the trestle, and setting the balk exactly over the legs, so that no strain is brought upon the cap.

c. By selecting for the cap a seasoned piece of bamboo from near the butt of a stalk, where the fiber is thick.

d. By using for the cap a fascine of split bamboo.

e. By introducing bearing pieces of thick bamboo between the cap and the ends of the legs.

f. By a combination of two or more of the above methods.

These trestles are so easily constructed that if built carefully they will usually meet all requirements, making it unnecessary to

resort to the heavier forms. When the men have had some experience in the work they can construct one of these trestles in about the same time that would be required to construct a timber trestle of equal size. The bamboo can be cut and handled more rapidly than timber, but the lashings are more elaborate and require more care.

The tools required to construct one of these trestles are: hand-axes, hand-saws, gabion knives, machetes, axes, wire cutting pliers. No. 14, A. W. G. is the most suitable size of wire.

STRENGTH TESTS.

Twenty small beams of bamboo were tested for transverse strength. The average fiber stress at the instant of failure was:

Seasoned bamboo, 17,000 pounds per square inch.
Green bamboo, 16,000 pounds per square inch.

The highest recorded test was 24,000 pounds and the lowest 10,000 pounds. Most of the specimens were quite near the average. Rupture was always preceded by a considerable deflection. The specimens were tested bark up and bark down, there being no difference of strength observable.

A number of specimens were tested for tensile strength by direct pull. The specimens were cut to a shape similar to a cement briquette, having a carefully measured minimum section. An astonishing strength was developed, as follows:

Seasoned bamboo, average tensile strength, 29,000 pounds per square inch
Green bamboo, average tensile strength, 23,000 pounds per square inch.

The highest recorded test was 34,000 pounds and the lowest 21,000 pounds. The fiber near the outside of the tree was found to be somewhat stronger than that near the center. The minimum cross section of the pieces tested was ⅛ inch square.

The test pieces for the compressive tests were ¼ inch to 3-16 inch square in cross section, and ½ inch high. The average strength in compression between two iron plates was as follows:

Seasoned bamboo, average compressive strength, 8,800 pounds per square inch.
Green bamboo, average compressive strength, 7,000 pounds per square inch.

As has already been noted, failure resulted from splitting of the fiber as well as direct compression. It is quite probable that specimens having a greater cross section would have developed a higher compressive strength. It is to be noted that the transverse fiber stress was computed by the formula for beams, $P=My/I$, in which y was assumed as half the depth of the beam, and M the bend-

ing moment of the known breaking load. The average depth (d) of the beams tested was 11-32 inch. The difference between the fiber stress resulting from the application of the formula, and the tensile stress as determined by actual tension, is accounted for by the considerable variation between the tensile and compressive strengths of the material.

The following is the result of the tests of small columns. These had square ends and were tested between two iron plates. The first series of five were tested bare. The second series were wrapped at each end with marline. The first failed by crushing and splitting at the ends, the last by buckling near the center.

Dimensions of column.			Kind of bamboo.	Breaking load.
Length.	Diameter.	Thickness of fiber.		
Inches.	*Inches.*	*Inches.*		*Pounds.*
12	1	$\frac{3}{16}$	Seasoned	1,280
$17\frac{1}{4}$	$1\frac{5}{16}$	$\frac{3}{16}$	Seasoned	3,440
18	$1\frac{1}{2}$	$\frac{3}{16}$	Green	2,340
$12\frac{1}{2}$	$1\frac{1}{32}$	$\frac{1}{8}$	Green	2,025
$13\frac{1}{8}$	$1\frac{1}{32}$	$\frac{1}{8}$	Half seasoned	2,220
18	$1\frac{1}{2}$	$\frac{3}{16}$	Half seasoned	2,430
$13\frac{1}{2}$	$1\frac{1}{8}$	$\frac{1}{4}$	Seasoned	3,800
$13\frac{5}{8}$	$1\frac{1}{8}$	$\frac{3}{16}$	Green	2,780
15	$1\frac{1}{4}$	$\frac{3}{16}$	Green	3,480
15	$1\frac{1}{4}$	$\frac{3}{16}$	Half seasoned	3,110
15	$1\frac{1}{4}$	$\frac{1}{4}$	Seasoned	2,360
$17\frac{1}{4}$	$1\frac{7}{16}$	$\frac{3}{16}$	Half seasoned	2,900

Failure of Coffer at Lock and Dam No. 48, Ohio River

BY

Maj. J. C. Oakes
Corps of Engineers; Member American Society Civil Engineers

At about 7 a. m. Monday, July 21, 1913, there occurred a failure of the main cofferdam at lock and dam No. 48, Ohio River. (For description of the coffer, see Professional Memoirs, Corps of Engineers, U. S. Army, Vol. V, No. 20.) This coffer was built 20 feet above low water and at the time of failure the river stage was 12.4 feet above low water, or elevation 337.5 (Sandy Hook datum), the water surface within the coffer was at elevation 316.5, the head being 21 feet.

The failure occurred at the passway in the lower arm. This passway was left to enable the contractor to pass floating plant in and out of the coffer at stages from 18 to 20 feet, and, as originally constructed, was 41 feet wide with bottom 6 feet below top of coffer, the area being closed by needles, as shown on the illustration.

After work had closed for the winter and the Government inspector withdrawn, this passway was torn out to remove some floating plant, and without the knowledge of this office was rebuilt with the top 2 feet lower than before. The top of the coffer, therefore, at this passway was only 12 feet above low water, needles being used to close the opening when required. On the outside of the coffer was a line of Wakefield sheet piles 26 feet long, with tops of piles from 10 to 12 feet above low water. These sheet piles extended about 10 feet below the bottom of the coffer. Sand was banked against the coffer to the height of the tops of the sheet piles.

In order to hold the banking against the coffer on the inside, the contractors had driven a line of sheet piles 60 or 65 feet away from the coffer with tops of piles at about low water elevation and fill had been placed sloping from the tops of these piles to the coffer at the elevation of the floor of the passway. This fill had been covered by gravel to prevent wash by seepage.

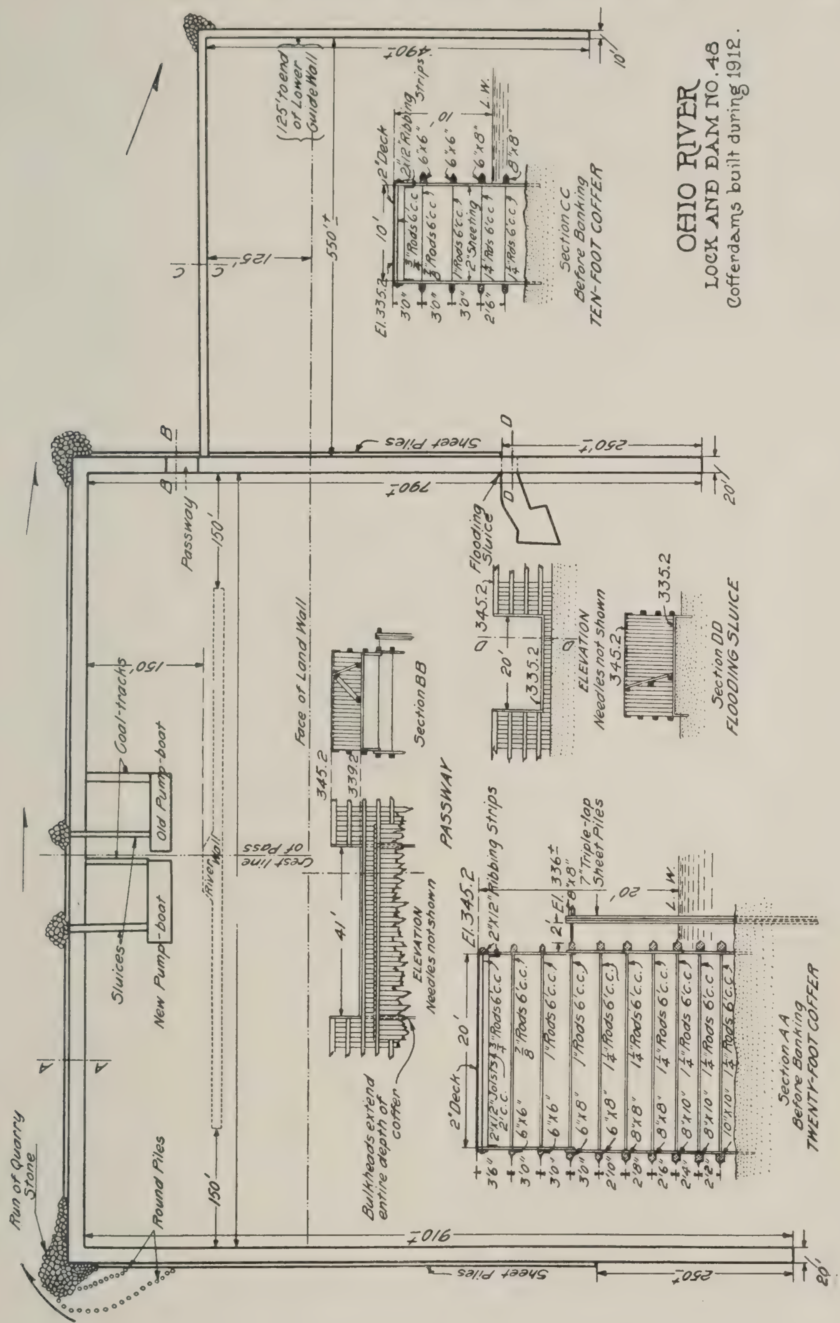
A small quantity of water, probably 1 cubic foot per second, has always collected behind the passway and flowed as a little stream into the excavated area, but as there were many of these little streams, some of them greater than this one, no fear of failure because of this seepage had been felt.

The Government inspectors lived on a quarterboat moored just below this passway and crossed the coffer constantly at this point, and no increase in the amount of seepage nor any movement of sand had been noticed.

The excavation was kept clear of water by three 15-inch centrifugal pumps on a pump boat, the latter resting on piles. The engineer of the pump boat noticed nothing unusual until 5.30 a. m. July 21, when he found the pumps were not holding the water surface at 315.5. He then changed his governors, increasing the speed of the engines and finally cut out the governors entirely, allowing the engines to run full speed. In spite of this, the water surface inside rose to 316.5 at 6.30 a. m. Shortly after this the men coming to work noticed a large leak near the passway; the alarm was given, and the pumps stopped at 6.45 a. m. At 7.01 a. m. there was evidently a blow-out, as the water was seen to suck down outside of the sheet piles; the sheet piles were lifted out, then the coffer lifted, and the break was completed. Before the inclosure was filled, about 250 feet of coffer had been washed away.

Through the gap thus created there were drawn four loaded coal barges, a barge of lumber, and one of round piles. The coal barges were rolled over and over, and were a total wreck, the pile barge was broken up, and the lumber barge was injured, but can be recovered and repaired. The pump boat was thrown off its pile foundation and somewhat injured and four pile drivers were submerged and probably injured to some extent, the amount of injury being unknown.

The contractor was about ready to place concrete for the river wall. The excavation was completed, all round piles and most of the sheet piles were driven and tracks on piles for derricks, cars, etc., were completed. A large amount of sand has been carried into the excavation, coal has been dumped about the heads of the piles and undoubtedly some of the tracks have been injured. It is estimated that the immediate money loss to the contractor will amount to between $10,000 and $15,000, but the accident may cause a much greater loss due to the delay, which is estimated at a month or six weeks, which may prevent the completion of the


OHIO RIVER
LOCK AND DAM NO. 48
Cofferdams built during 1912.
Section AA
Before Banking
TWENTY-FOOT COFFER
Section CC
Before Banking
TEN-FOOT COFFER
Section BB
PASSWAY
Section DD
FLOODING SLUICE
ELEVATION
Needles not shown
Face of Land Wall
Sheet Piles
Round Piles
Run of Quarry Stone
New Pump-boat
Old Pump-boat
Sluices
Coal-tracks
Passway
Flooding Sluice
Bulkheads extend entire depth of coffer

work inside the coffer this season and make necessary the unwatering again next year. It is hoped, however, that further investigation when the river falls will show less damage than is anticipated.

Several causes of the failure can be suggested, and it is probable that they all had a bearing; their relative importance, however, can only be guessed at. These probable causes are: (a) The weight of the passway coffer was probably about 1,000 pounds per square foot less than it would have been if it had been built to full height; (b) in rebuilding the passway, good connection between the old and new sheet piles and the sheeting of the coffer may not have been obtained; (c) the material used for filling the passway coffer when rebuilt may have contained a large proportion of silt as the silt deposit in the coffer during the winter was very heavy; (d) the seepage probably gradually increased with the increasing head until, during the night, the little stream began to carry out sand from beneath the coffer and it also probably cut down into the inside banking until the limiting plane of saturation was reached when the blow-out occurred.

It is certain that there was a considerable increase of flow during the night from this and probably other seepage streams, and that at 6.30 a. m. there was a bad leak under the passway, and when the blowout took place the whole mass became suddenly fluid and lifted the sheet piles and the coffer.

General George Brinton McClellan

Major-General George Brinton McClellan was born in Philadelphia, December 3, 1826. His father was a bold and successful surgeon and founder of the Jefferson Medical College in Philadelphia, in which he became professor of surgery and a popular lecturer.

George Brinton McClellan entered the Military Academy in 1842 at the age of sixteen and graduated No. 2 in his class in 1846. He was assigned to the Corps of Engineers and immediately ordered to Mexico, where he served as a lieutenant of Engineers under General Scott. For his work at Contreras and Cherubusco he was brevetted first lieutenant and later, for gallantry at Chapultepec, he was brevetted captain. Following the Mexican War he was detailed as assistant instructor at West Point and later employed in explorations and surveys in the southwestern part of the United States and on Pacific Railroad surveys in the northwest. In 1854 he was promoted to captain of Cavalry, and was later one of three officers chosen to study the war in the Crimea and to report on European armies. His report "On the Armies of Europe" was published in 1861, although he had resigned his commission in 1857 to become chief engineer and later vice-president of the Illinois Central Railroad, which positions he held until 1860. Following this he became General Superintendent of the Mississippi and Ohio Railroad and, at about the time of the breaking out of the Civil War, he was made president of the eastern branch of that road.

Throughout all his life General McClellan had the greatest capacity for making strong friends, so that it was only natural he should be well known in Ohio when the war broke out and that he should be made a major-general of militia by the Governor of Ohio. Ohio was then, as now, one of the great states in the Union, and what she did was noticed by the Government at Washington. Besides this General Scott, who commanded the Union forces at the breaking out of the Civil War, looked upon McClellan with a great deal of favor. Accordingly, on May 14, 1861, he was made a major-general in the United States Army and put in command of

the Department of the Ohio; shortly after this in a brief, but very fortunate campaign, the troops under General McClellan drove the Confederates in western Virginia across the mountains, thereby laying the foundation for the present State of West Virginia (admitted to the Union in 1863). While this short campaign involved but very small forces and resulted in only slight losses on either side, yet, coming as it did just prior to the disastrous defeat of the Union forces at Bull Run, it made General McClellan the foremost military figure in the United States. It is of interest at this point to recall that, while General McClellan was assigned to the Corps of Engineers upon graduation and served in that Corps until 1854, he was in the latter year promoted to be a captain of Cavalry. In the following year he designed the saddle that is still the standard in the service of the United States. Considering his wide and varied training, his study of European armies and the War of the Crimea, together with his brilliant personality, it is not surprising that following the defeat at Bull Run he was hailed as the "Little Napoleon" who should speedily crush the revolt of the Southern States.

Late in July of 1861 he arrived in Washington and was placed in command of the Union forces that had been engaged in the Battle of Bull Run, together with all others that had arrived in the National Capital after that date. In November General Scott, at his own request, was retired from the position of General-in-Chief and General McClellan appointed thereto. It is worthy of remark that at this time General McClellan was only thirty-four years old, being three years younger than Gen. Thomas J. (Stonewall) Jackson, who graduated seventeen in the same class in which General McClellan graduated number two.

There is not space and it is not considered desirable to attempt here to enter into any detailed account of General McClellan's services to the Union during the Civil War. In fact, General McClellan's portrait has been chosen as a frontispiece for the present number of the Professional Memoirs for the reason that he was an Engineer officer and that the Battle of Antietam was fought September 17 and 18, 1862, just fifty-one years ago this month. General McClellan has been one of the most harshly criticised of the generals of the Civil War but, unlike some others who have shared a nearly similar fate in that regard, he has also been widely heralded as one of the greatest, if not the greatest, general of that war. Somewhere between these two extremes lies the proper value of his

services. It seems, however, to be conceded by most critics to-day that his fame rests most securely on his ability as an organizer. The forces in and around Washington were in a very bad state of disorganization and demoralization when he took charge; in a few months he so thoroughly organized what was afterward known as the Army of the Potomac that, notwithstanding numerous defeats, it was never disorganized and finally formed the greater part of the army that under Grant forced the surrender of Lee at Appomattox.

Perhaps one of the greatest mistakes that General McClellan made when he was sent to Washington was to establish his headquarters in that city and not with the troops he commanded, the latter being stationed a few miles away in the vicinity of Alexandria, Va. Having the most genial and lovable personality, he was the social lion of Washington and the one man on whom more publicity probably was showered than on the President himself.

By early fall in 1861, McClellan had an army of about one hundred thousand men fairly well equipped and probably as thoroughly organized as was possible, considering the time available. In this work of drilling he had the assistance of many of the ablest officers of the Army, including Gen. William Tecumseh Sherman, who lived on the ground with the troops and worked incessantly to train himself as well as the men under him.

At this time, just as before the Battle of Bull Run, the public began to clamor for a forward movement against the Confederates, who were being recruited and drilled in Virginia not many miles away from the Union Army. Not only did General McClellan fail to yield to this desire of the country, but, as the demand for an advance became more imperative, he apparently became more opposed until winter finally set in in earnest, making the roads impassable and rendering impossible any forward movement until the following spring.

For the first few weeks after assuming command of the forces in and about Washington, General McClellan was all elation and highly pleased with everything and everybody; later, he began complaining of the inadequacy of the number of troops and supplies furnished him, at the same time greatly overestimating the strength of the enemy. Early in April in 1862, in accordance with his plan, the Army was transported to Fortress Monroe on the peninsula between the James and York rivers. Here he allowed a comparatively small force to hold his entire army for nearly three weeks

while the Southern forces were busy concentrating their own forces and preparing to resist his advance on Richmond. Then followed his slow advance and finally the shifting of his base of supplies from White House Landing on the Pamunkey River to the James River. This latter was really a brilliant feat and one can not help thinking that had he used the same daring in devising and pushing offensive campaigns against the enemy, he would have proven to be what some of his enthusiastic admirers claim for him, the greatest general of the Civil War.

McClellan and his army remained inactive near Harrisons Landing on the James River for six weeks or until the middle of August, when its removal to Washington was begun. It arrived at the latter place during the final stages of the Battle of Manassas or Second Bull Run, when the Union Army under Pope was badly defeated. Pope was at once relieved by McClellan, who was put in charge of all the troops in the vicinity of Washington.

This was September 2d, just two weeks before the Battle of Antietam, where Lee's army was so badly cut up by McClellan that it was forced to retreat twenty-four hours after the close of the battle.

This was the first great battle in the eastern theater of the war where the Union forces were able to drive the Confederates from the field. The victory was most opportune and had consequences only a little, if indeed any, less important than the victory of Meade a year later at Gettysburg. It stopped the invasion of the Northern States by Lee; it checked the sentiment growing in Europe for the recognition of the Southern Confederacy and gave President Lincoln an opportunity to promulgate his famous emancipation proclamation.

And yet, while giving Gen. McClellan due credit for the victory, the feeling remains that had he moved with celerity and promptness when he learned that Lee had divided his forces he could have fallen upon those divided forces in detail and have won a decisive victory. After the battle McClellan's pursuit was slow and cautious and, while he maneuvered so as to get his army into a very favorable position in Virginia and to slowly drive Lee back, his work was not satisfactory to the Government at Washington and he was relieved on November 7.

Upon his relief from the command of the Army he was ordered to Trenton, N. J., and took no further part in the war. He remained in and around New York awaiting orders until November 8, 1864,

on which date he was defeated for the Presidency and his resignation from the Army accepted. Immediately after his defeat he sailed for Europe, where he remained until 1868, when he returned and took charge of the construction of what was known as the Stevens steam floating battery at Hoboken, N. J. In 1870 he was appointed Chief Engineer of the Department of Docks of New York City, but resigned the position in 1872. In 1878 he was elected Governor of New Jersey, but declined a renomination when his term expired in 1881. During his term as Governor he took an active interest in many subjects, including education and the State militia. In 1881, pursuant to an act of Congress, he was appointed a member of the Board of Managers of the National Home for Disabled Soldiers and continued in that position until his death, which occurred very suddenly at his home in Orange, N. J., October 29, 1885.

In addition to the above, he served as consulting engineer on many large engineering works between 1868 and the date of his death.

In studying McClellan's life and history certain qualities of mind and spirit stand out in bold relief above all the fog of controversy. In the first place he was not only brilliant intellectually and popular with the general public as heretofore mentioned, but he had a wonderful capacity for gaining the enduring affection of the troops serving under him. Notwithstanding his failure to follow up his victories or to inflict a severe defeat upon the enemy, he was unquestionably the best loved officer that ever served with the Army of the Potomac, and many readers of the MEMOIRS will probably remember that in one of the interviews with veterans at the recent reunion at Gettysburg, one stated that during the early part of the Battle of Gettysburg a rumor became current that McClellan was again in command of the Army and that this rumor inspired the greatest possible confidence among the men.

This quality and his ability as an organizer are agreed to to-day by all sides to the controversy over his ability as a general. Indeed, it is admitted as doubtful if any other man could have so organized the Army of the Potomac as to enable it to keep its morale and fighting spirit through the many reverses it suffered between the spring of 1862 and the surrender of Lee in 1865.

His principal fault would appear to have been that he never learned what Grant says he learned in some of his first skirmishes with the enemy; that is, that if he were having difficulties with

supplies and transportation, the enemy was probably having equal or greater difficulties, and that if he (Grant) were afraid of the enemy the enemy was probably equally afraid of him and that the one who put on the boldest front and struck first and hardest would, in all probability, win.

Finally, whatever theoretical conclusions may be arrived at as to his ultimate abilities as a great commander in the field, the fact remains that within a few months after his relief from command the Army of the Potomac suffered two bloody and disastrous defeats; first at Fredericksburg and then at Chancellorsville. No injuries comparable with these two were ever inflicted upon the Army under McClellan's command and in all probability the Union forces would have been in far better condition for the great Battle of Gettysburg in 1863 had McClellan continued in command. Indeed, as Palfrey says in his book on the Antietam and Fredericksburg campaigns, "There are strong grounds for believing that he (McClellan) was the best commander the Army of the Potomac ever had."—A. A. F.

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Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
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(8) Cornell Civil Engineer.
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*Now combined under title: Concrete-Cement Age.

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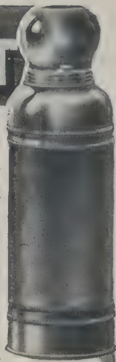

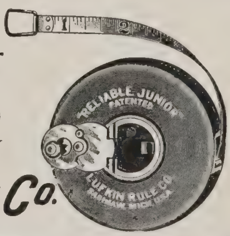

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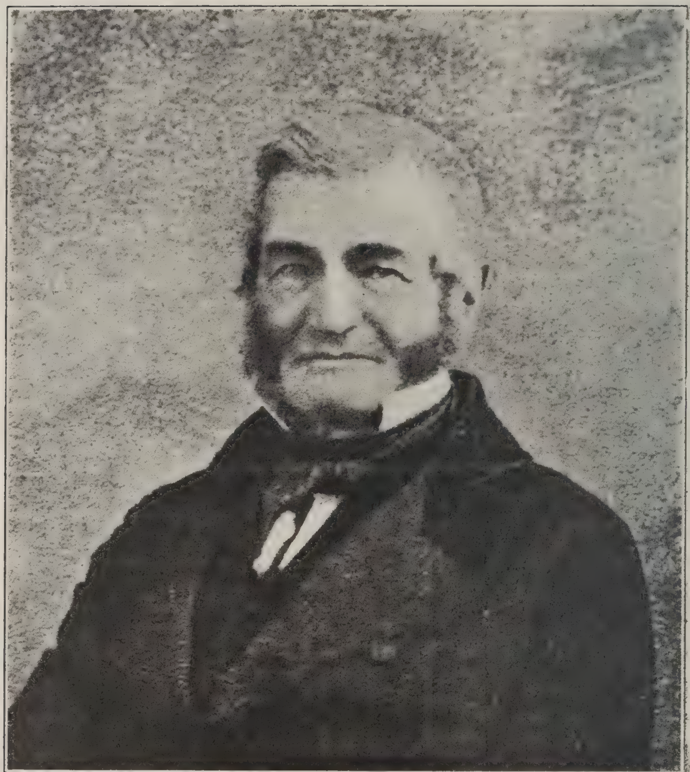

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The Colbert Shoals Canal, Tennessee River, Alabama

BY

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INTRODUCTORY.

It was the intention of the writer when beginning the preparation of an article on the Colbert Shoals Canal to give only a brief description of the development of the plans for the canal and then to go fully into methods of construction and types and arrangement of plant, and to give such other detailed information as might be of interest in connection with an important engineering work of this character, including unit costs for the different classes of work. Study of office records has resulted, however, in increasing the importance of that part of the article treating of the development of the final plans and in minimizing the importance of the other subjects mentioned above. The construction of the canal extended over a period of twenty years—from the beginning of work on the lift lock in the fall of 1891 to the completion of the canal in December, 1911. Later on in this article there is a list of appropriations made for this improvement, with the date of each appropriation. From this list it will be noted that in the earlier history of the work the appropriations were small and far apart, which explains the unusual length of time consumed in the prosecution of the work. However, the long duration of the work, coupled with many changes in personnel, operated to give excellent opportunity for full study of the details of the plans in the light of additional data obtained during the progress of the work, and also made possible changes in those plans where later information indicated that changes were desirable. Had Congress provided funds in the beginning in keeping with the magnitude of the work, the progress toward completion would have been too rapid to permit the studies necessary for making the plans conform to the additional information of hydraulic conditions obtained from time to time, in which

event the project for the improvement of the Colbert Shoals section of the Tennessee River would not have met with the changes which it has in fact undergone. The numerous changes in plans have caused some adverse comment, but a careful study of the voluminous papers containing the discussions which preceded the changes leads the writer to the conclusion that each of them came as a logical result of the consideration of the further data obtained, and that each of the changes was in the interest of greater ease of navigation, of safer structures, or of economy of construction. That this is a fact does not, however, argue for slowness in making appropriations for engineering works in order to allow additional time for detailed study and changes of plan, for even in this case the slow manner in which the money has been supplied has caused the improvement to cost more than it would have cost if it had been rapidly pushed to completion. Moreover, the delay of fourteen or fifteen years in making this improvement available for boats has caused more injury to navigation than can be offset by the moderate benefits resulting from its being completed according to a plan somewhat better than the original.

For the above reasons it is believed that a full discussion of the various modifications of the project and the objects expected to be attained thereby will prove more interesting and instructive than would the limited and incomplete discussions of methods of construction, plant lay-out, and unit costs, possible to be extracted from the incomplete office records. The literature on the subject of the plans for the canal and the changes therein is nearly complete, but there is very little information on file as to construction methods or as to costs. Furthermore, because of the long period in which the work was under way, many of the assistants connected with the work have left the service or have died, so that little information is obtainable from that usually available source. Consequently, this paper is of necessity confined rather to the development of the final plans of the work than to methods of construction, description of plant, and details of cost.

DESCRIPTIVE.

Colbert and Bee Tree shoals, which may be considered as forming one continuous obstacle to navigation, extend from a point 21.4 miles below Florence, Ala., to Riverton, Ala., a distance of 8.6 miles; and there is in this 8.6 miles a total fall at low water of about 26 feet. The Colbert Shoal is 4.2 miles in length, with a total

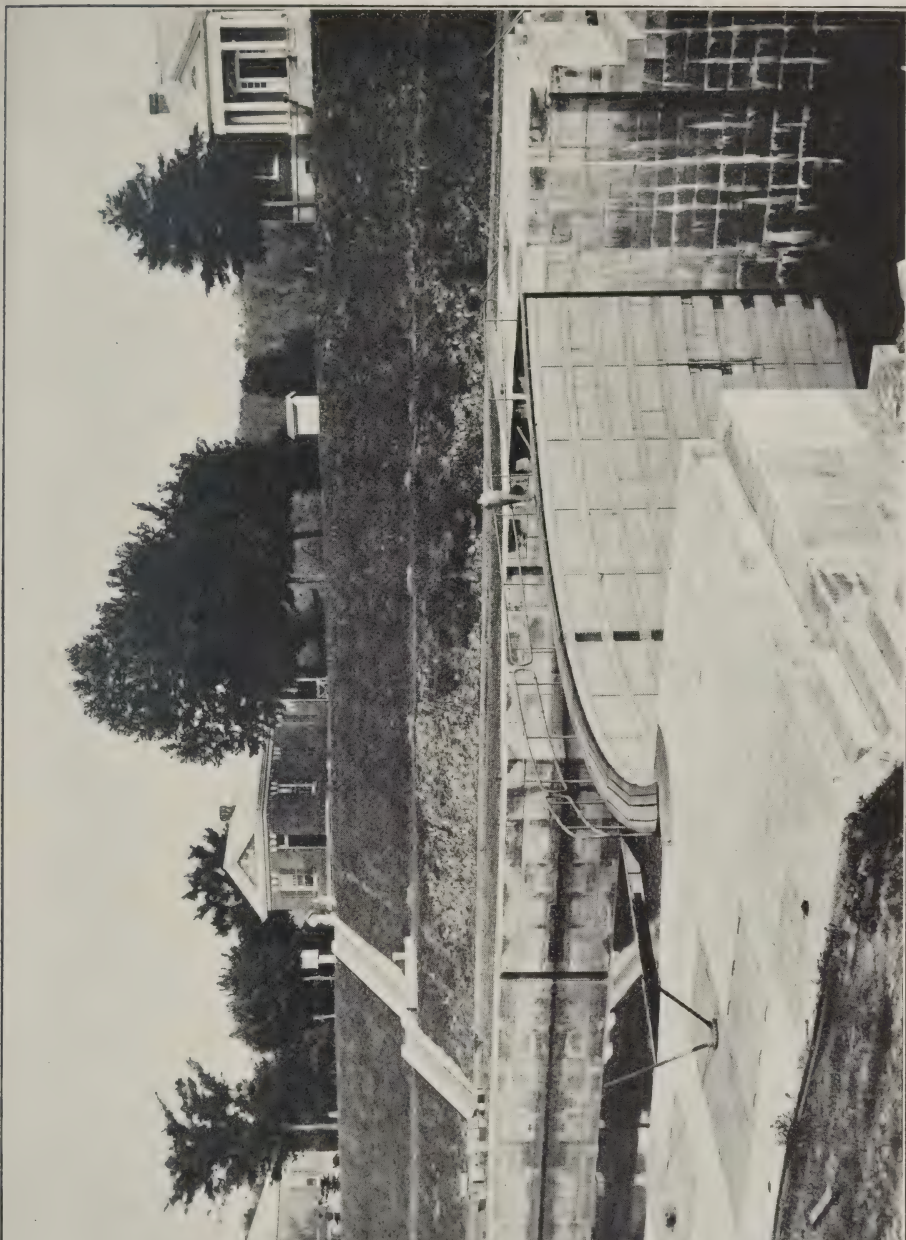

Fig. 1. Artificial mound, with buildings, Colbert Shoals Canal, Tennessee River, June 17, 1913; looking across canal.

fall of 14.5 feet and with a maximum slope of 3.15 feet per mile. The pool between the two shoals is 1.95 miles long and has a total fall of 0.8 foot. Bee Tree Shoal is 2.45 miles long, and has a total fall at low water of 10 feet and a maximum slope of 4.85 feet per mile.

The minimum depth in the channel over the shoals at ''ordinary'' low water is about 30 inches, with rock bottom; but at ''extreme'' low water the minimum channel depth is not more than 14 or 15 inches. The channel has less than 5 feet depth for about three-fourths of the total length of the shoal section. At all times except during high water the current is so swift as to materially interfere with navigation, even when there is sufficient depth in the channel. Generally, it may be said that boats of 6-foot draft can safely go down over the shoals when the Riverton gage* reads 16.0 feet or over; that up-bound boats of 6-foot draft may pass the shoals when this gage reads 17.5 feet or over; and that boats of 3-foot draft can pass down at a reading of 6.0 feet. From the table printed herein the navigable season can be ascertained for each year of

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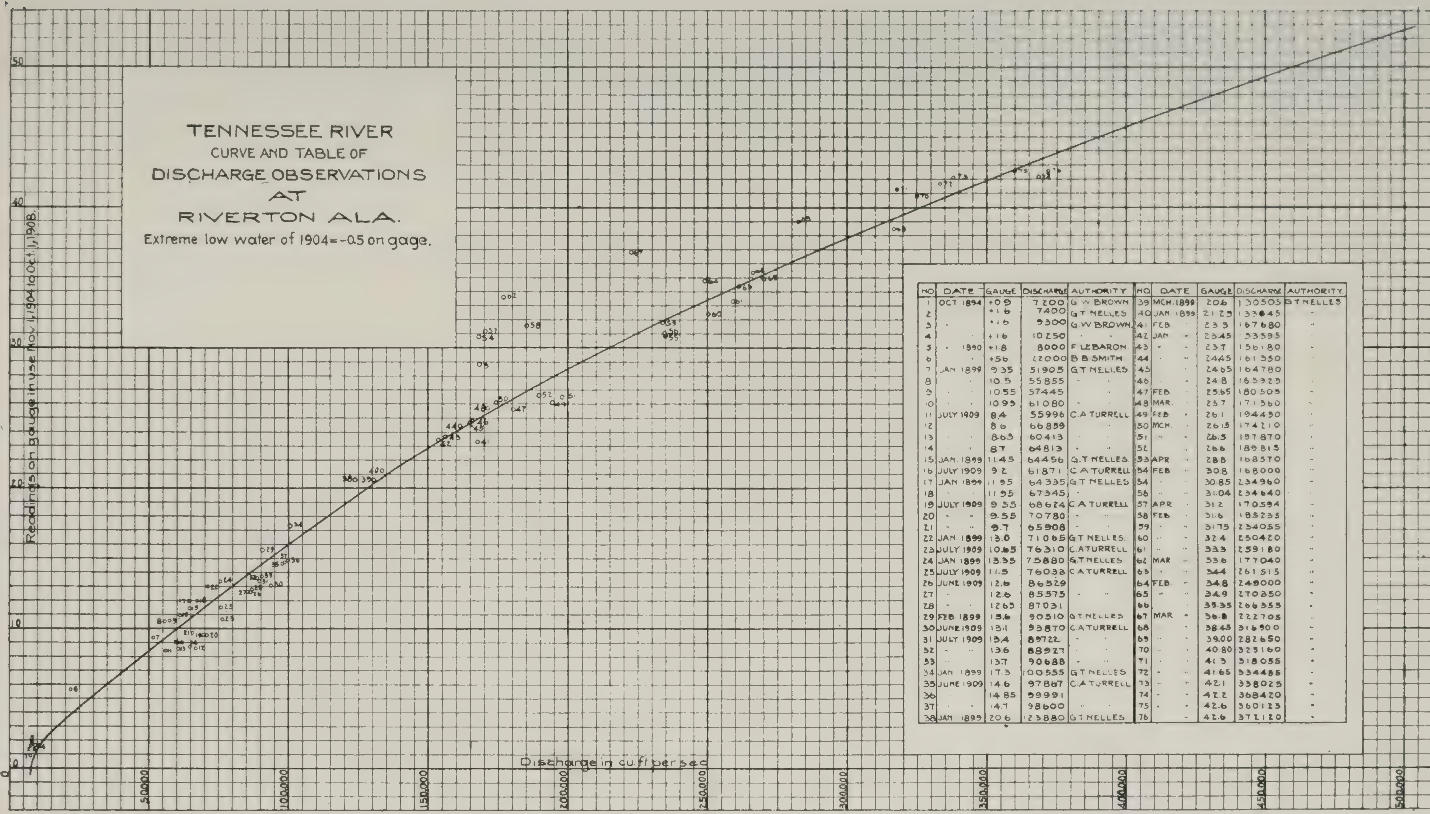
*There have been three Riverton gages: That of 1890 had zero 359.92 above sea level, or 10 feet above ''canal datum.'' This gage was formerly designated as ''gage No. 4'' and is frequently referred to in this article by that designation. A new gage was established on November 1, 1904, about 2,000 feet upstream from gage No. 4, with zero 357.92 above sea level, or 8.0 feet ''canal datum.'' In the table it is the 1904 gage which is referred to, and throughout this article where any reference is made to ''Riverton gage'' the 1904 gage is meant. On October 1, 1908, readings began to be made on the present gage, which is cut on the lower approach wall of the lock and is in close proximity to the 1904 gage. The zero of the present gage is at sea level reference 351.42, and at ''canal datum'' reference 1.5, being 0.25 foot above top of the lower miter sill.

Three other gages have been maintained on the shoal section and are mentioned occasionally in this article. Gage No. 1 was at the head of the Colbert Shoals, about 100 feet above the head of the canal, and had zero at 34.02 on ''canal datum;'' gage No. 2 was near Boones Branch and had zero 20.55 feet above ''canal datum;'' and gage No. 3 near the head of Bee Tree Island, with zero at 19.3 ''canal datum.''

''Canal datum'' refers to a plane assumed as datum at the beginning of the work and since determined to have sea level elevation of 349.92, M. R. C. determination from the Gulf. Throughout this article references, where not otherwise stated, are given as to ''canal datum.''

Relation of ''canal datum'' to water surfaces in the river and canal are as follows: Assumed low water elevation at head of canal, 33.5; actual extreme low water elevation, 33.92; assumed low water elevation at foot of canal, 8.0; actual extreme low water elevation, 7.5; extreme high water at head, 64.0, and at foot, 60.3.



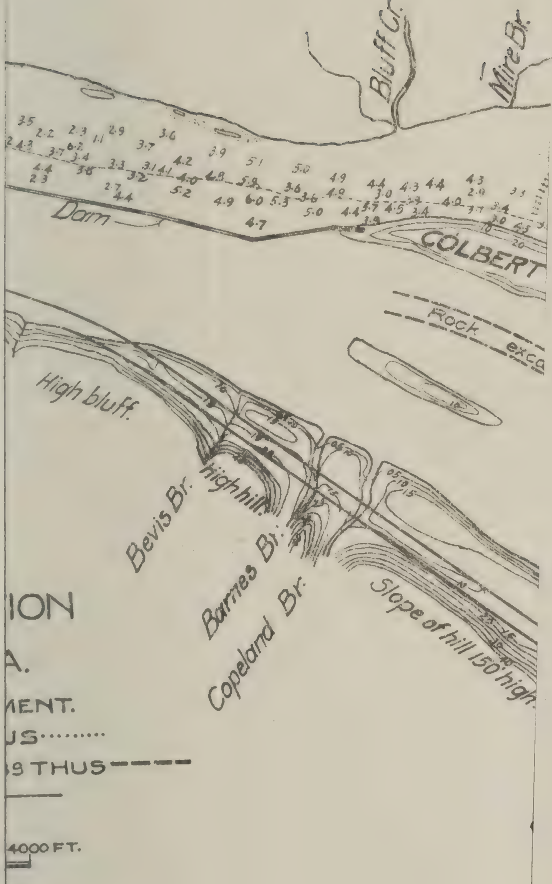
NO	DATE	GAUGE	DISCHARGE	AUTHORITY	NO	DATE	GAUGE	DISCHARGE	AUTHORITY
1	OCT 1894	+0.9	7200	G W BROWN	39	MCH. 1899	20.6	130505	G T NELLES
2		+1.6	7400	G T NELLES	40	JAN. 1899	21.25	133645	"
3	"	+1.6	9300	G W BROWN	41	FEB "	23.3	167680	"
4	"	+1.6	10250	" "	42	JAN "	23.45	133595	"
5	" 1890	+1.8	8000	F LEBARON	43	" "	23.7	156180	"
6	"	+5.6	22000	B B SMITH	44	" "	24.45	161350	"
7	JAN. 1899	9.35	51905	G T NELLES	45	"	24.65	164780	"
8	"	10.5	55855	" "	46	" "	24.8	165925	"
9	" "	10.55	57445	"	47	FEB. "	25.65	180505	"
10	" "	10.95	61080	" "	48	MAR. "	25.7	171360	"
11	JULY 1909	8.4	55996	C.A TURRELL	49	FEB "	26.1	194450	"
12		8.6	66859	"	50	MCH. "	26.15	174210	"
13	" "	8.65	60413	" "	51	" "	26.5	197870	"
14	" "	8.7	64813	" "	52	"	26.6	189815	"
15	JAN. 1899	11.45	64456	G.T NELLES	53	APR "	28.8	168570	"
16	JULY 1909	9.2	61871	C A TURRELL	54	FEB "	30.8	168000	"
17	JAN 1899	11.95	64335	G T NELLES	54	" "	30.85	234960	"
18	" "	11.95	67345	"	56	" "	31.04	234640	"
19	JULY 1909	9.55	68624	C.A TURRELL	57	APR "	31.2	170594	"
20	" "	9.55	70780	" "	58	FEB. "	31.6	185235	"
21	" "	9.7	65908	" "	59	" "	31.75	234055	"
22	JAN. 1899	13.0	71065	G T NELLES	60	" "	32.4	250420	"
23	JULY 1909	10.65	76310	C.A TURRELL	61	" "	33.3	259180	"
24	JAN 1899	13.35	75880	G.T NELLES	62	MAR "	33.6	177040	"
25	JULY 1909	11.5	76038	C A TURRELL	63	" "	34.4	261515	"
26	JUNE 1909	12.6	86529		64	FEB "	34.8	249000	"
27	"	12.6	85575	" "	65	" "	34.9	270350	"
28	" "	12.65	87031		66	"	35.35	266355	"
29	FEB 1899	15.6	90510	G T NELLES	67	MAR "	36.8	222705	"
30	JUNE 1909	13.1	93870	C.A TURRELL	68	"	38.45	316900	"
31	JULY 1909	13.4	89722	" "	69	" "	39.00	282650	"
32	" "	13.6	88927	" "	70	" "	40.80	323160	"
33	" "	13.7	90688	" "	71	" "	41.3	318055	"
34	JAN 1899	17.3	100555	G T NELLES	72	" "	41.65	334485	"
35	JUNE 1909	14.6	97867	C.A TURRELL	73	" "	42.1	338025	"
36	"	14.85	99991	"	74	" "	42.2	368420	"
37	" "	14.7	98600	" "	75	" "	42.6	360125	"
38	JAN 1899	20.6	123880	G T NELLES	76	"	42.6	372120	"

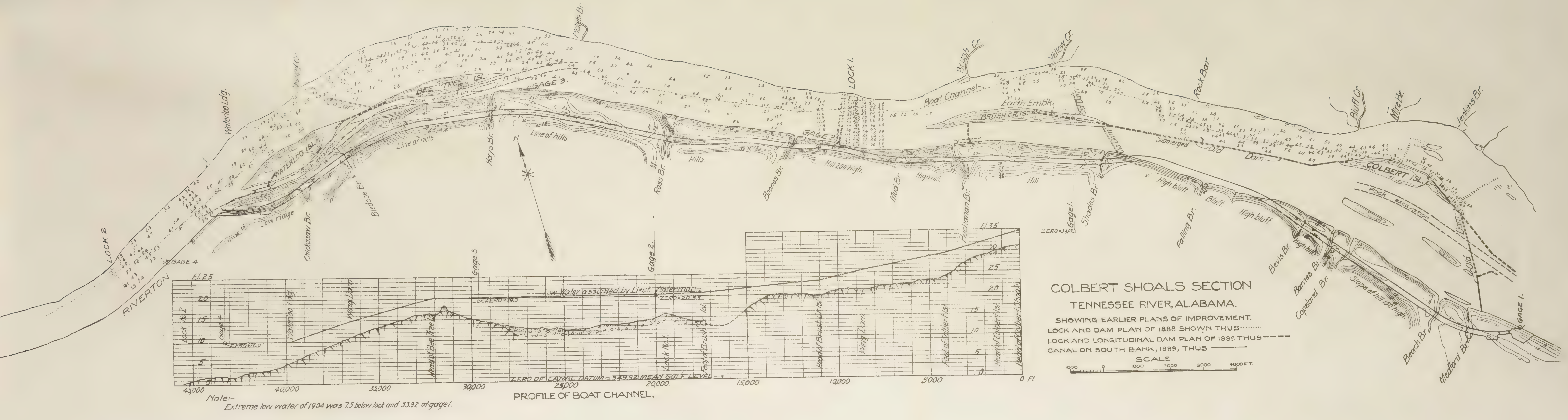
Fig. 2.

the record. This table shows that 3-foot draft can be carried downstream on the average of only about six and one-half months per year, and that upbound 6-foot draft can be carried past the shoals for only about one and one-half months per year. It is very likely that careful pilotage and well-powered boats might increase the periods stated for the different drafts, but not to any considerable extent.

In Plate I there is given a map of the shoals, from which it may be noted that the widths of the river in the shoal section are from 2,000 to 4,000 feet, while just below the width is not more than 1,300 feet. Even this latter figure is somewhat greater than the average width, which is about 1,000 feet, for the 226 miles between Riverton and the mouth of the Tennessee. The map shows the four islands (Colbert, Brush Creek, Bee Tree, and Waterloo) which, with the smaller islands and towheads, divide the river into several channels or chutes. While a much fuller description might be given of this shoal section, it is perhaps sufficient to add that these shoals in their natural condition constituted the most serious obstruction to navigation in the 255 miles of river below the foot of the Muscle Shoals section, and formed an almost complete barrier to upbound navigation for about five and one-half months per year. However, this obstruction was not as complete a one as that at the Muscle Shoals section, which was at all times impassable except for small skiffs.

The surveys of these shoals included discharge observations, made at various times and at different river stages—in 1890 by F. L. Baron and B. S. Smith, in 1894 by S. W. Brown, in 1899 by G. T. Nelles, and in 1909 by C. A. Turrell. From the data obtained from these measurements there was prepared a discharge curve, which is reproduced in Plate II. Because of the limited number of observations at extreme high and extreme low water, the curve is not much more than an approximation at its upper and lower limits, but is probably nearly correct for intermediate points. The curve indicates that at the lowest recorded Riverton gage reading of —0.5 the discharge was approximately 7,500 second-feet, and at the highest recorded gage reading of 52.3 the discharge was about 495,000 second-feet. This lowest reading of —0.5 occurred in October, 1904, while the maximum gage reading was in March, 1897. The watershed of the Tennessee River above Riverton is approximately 30,850 square miles, with an average annual rainfall of about 48 inches. (For details of methods used in making


Bluff Cr.
Mire Br.
Dam
COLBERT
Rock exca
High bluff.
Bevis Br.
high hill.
Barnes Br.
Copeland Br.
Slope of hill 150' high.
ION
A.
MENT.
US.........
S THUS
4000 FT.



COLBERT SHOALS SECTION
TENNESSEE RIVER, ALABAMA.
SHOWING EARLIER PLANS OF IMPROVEMENT.
LOCK AND DAM PLAN OF 1888 SHOWN THUS
LOCK AND LONGITUDINAL DAM PLAN OF 1889 THUS
CANAL ON SOUTH BANK, 1889, THUS
SCALE
1000 0 1000 2000 3000 4000 FT.
LOCK 2
RIVERTON
GAGE 4
Waterloo Ldg.
WATERLOO ISL.
Low ridge
Chickasaw Br.
Bledsoe Br.
BEE TREE ISL.
GAGE 3
Rock excavation
Line of hills
Hoys Br.
Line of hills.
N
Picketts Br.
Ross Br.
Hills.
Boones Br.
GAGE 2
LOCK 1.
Hill 200' high.
Mud Br.
High hill
Buchanan Br.
Brush Cr.
Boat Channel
Earth Embk.
BRUSH CR. ISL.
Yellow Cr.
Old Dam
Hill
Gage 1.
ZERO = 34.02
Shades Br.
Rock Bar
Submerged Old Dam
High bluff
Bluff
Falling Br.
High bluff.
Bevis Br.
High hill
Barnes Br.
Copeland Br.
Slope of hill 150' high
Beach Br.
Medford Br.
GAGE 1.
Old Dam
Rock excavation
COLBERT ISL.
Bluff Cr.
Mire Br.
Jenkins Br.
El. 25
El. 35
Lock No. 2
Gage 4
ZERO = 10.0
Waterloo Ldg.
Wing Dam
Gage 3
Head of Bee Tree Isl.
ZERO = 18.3
Low Water assumed by Lieut. Waterman
ZERO = 20.55
Gage 2.
Lock No. 1
Foot of Brush Cr. Isl.
Head of Brush Cr. Isl.
Wing Dam
Foot of Colbert Isl.
Head of Colbert Isl.
Head of Colbert Shoals
ZERO OF CANAL DATUM = 349.92 MEAN GULF LEVEL
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5000
0 Ft.
PROFILE OF BOAT CHANNEL.
Note:-
Extreme low water of 1904 was 7.5 below lock and 33.92 at gage 1.

Fig. 3. Excavating canal prism by Armstrong Steam Excavators. Station 210-260; looking upstream.

discharge observations, see Report of the Chief of Engineers for 1899, pages 2277 to 2286.)

IMPROVEMENT BY REGULATION.

Some work was done toward improving these shoals under a small appropriation made in 1827, but no record is available which gives either the amount of the expenditures or the nature and extent of the work done. In 1868, 1869, and 1870 there were made from the appropriations for the improvement of the Tennessee River allotments aggregating $30,000 for the purpose of improving Colbert and Bee Tree shoals "with a view to obtaining a 3-foot low water depth in channel." Unfortunately, the records are quite incomplete as to this earlier work of improving conditions at these shoals, but it is known that contracts were let and work was begun on the project for obtaining a 3-foot channel depth. The plans for this improvement can not be found, but it is apparent from such maps as are at hand that the increase of depth was to be obtained primarily by works of contraction aided by rock excavation. The extent of the work assumed to be necessary when the work was begun can not be stated, but it is very likely that the contraction works (see Pl. I) as actually constructed represent about what was planned for the purpose of contracting the channel. The various references in old reports to rock excavation indicate that this item of the improvement was not of any considerable extent, but no figures can be given as to quantities. It seems, however, that the bulk of the rock for dikes had to be obtained from quarries. The works shown on the map as having been constructed for the purpose of contracting the channel are (1) a closing dike connecting the head of Colbert Island with the left bank of the river; (2) a longitudinal training dike, running downstream from Colbert Island; (3) a closing dike connecting the head of Brush Creek Island with the left bank of the river; (4) a spur dike projecting toward the channel from the head of Brush Creek Island; and (5) a spur dike running diagonally out from the foot of Bee Tree Island toward the right bank. These works contracted the channel on Colbert Shoals to approximately one-third of its original width, and on Bee Tree Shoals to approximately two-fifths of the former width. In the report of the district officer for 1873, it was stated that it might possibly be necessary to construct a spur dike from the right bank out to the "Rock Bar" (probably about 2,000 feet above Brush Creek Island) or in that vicinity, but this was never done.

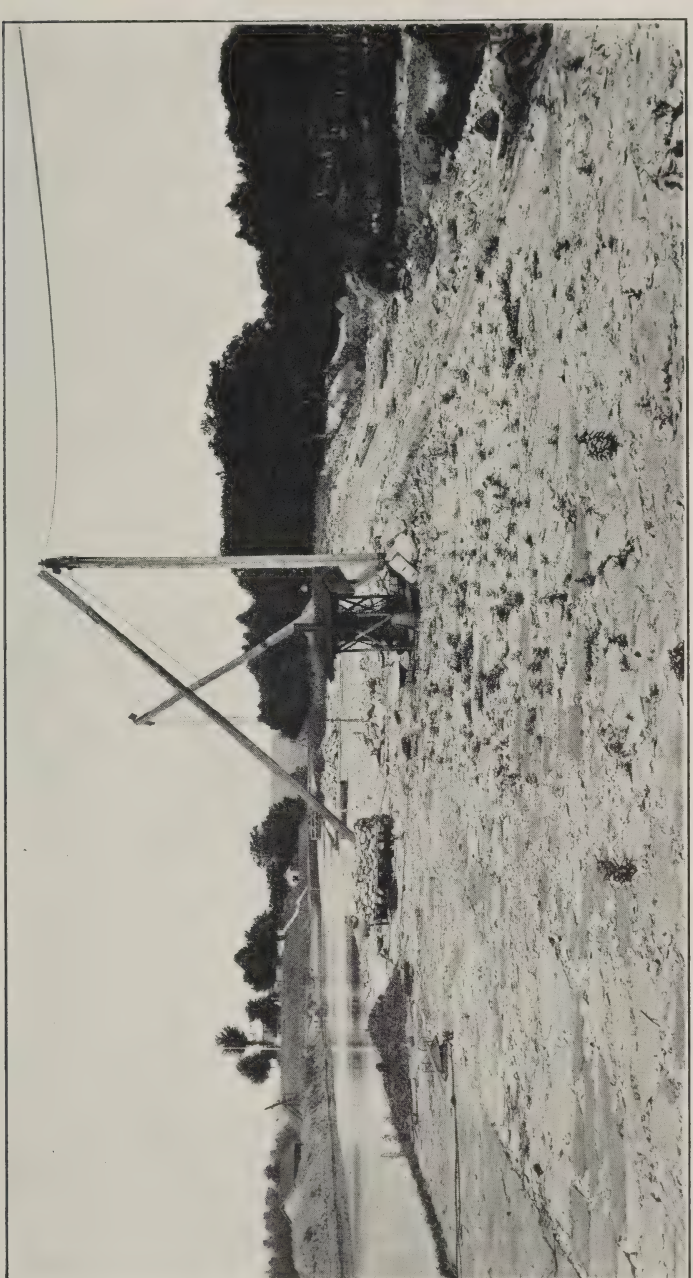


Fig. 4. Waste weir No. 1, from upstream end; showing lock and mound.

While, as stated above, the office records are quite incomplete as to details and progress of the work of improvement by regulation, there are a number of statements scattered through the annual reports which are of interest in connection with the improvement of these shoals by regulation and the results secured thereby.

Work under the contracts was prosecuted very slowly and, finally, after several extensions of the contracts, they were terminated in 1872, after which the work was prosecuted by the hired labor method. In the 1873 report the district officer calls attention to the very satisfactory results obtained by the hired labor method and states, "while the work is better done in every respect, and is done promptly, and with no loss of time; while the workmen receive the best wages given in this region, are rationed, and work but eight hours a day, they have actually done more work per day than the contractors ever accomplished, and have done it on the average at much lower rates, both in the removal of rock from the channels and in the construction of dams." Again, in 1874, is the following: "Nowhere on the river has the difference between the work done by hired labor and that done under contract been more conspicuous than here. While the dams built under the former system are regular, solid and tight, those built under the latter system are crooked, loose, and weak, and must without exception be rebuilt." This latter statement seems to have been correct, as there are several mentions in later reports of damage to the contract dams and none of damage to those built by hired labor. Apparently, in the years following 1873 all of the old contract dams were either rebuilt or extensively repaired and strengthened. The 1874 report continues: "The proper construction of these dams, in the first instance, which could have been secured by the employment of hired labor, would have resulted in a saving of all it will cost now to put them in order, which will probably amount to one-half their original cost."

Before the end of the fiscal year 1874 all of the rock excavation and the dike construction contemplated when the contracts were let had been completed, and after that time it appears that the dikes were not extended nor were any new ones constructed, although considerable work was done from time to time in the repair of the old contract dikes. Occasionally afterwards there were excavated small quantities of rock from places where experience showed that the available depth in the channel could be used with much greater safety to navigation if projecting points or lumps were removed. After the year 1874 the allotments for the improvement of

these shoals were infrequent and of small size, until the canal project was taken up by Congress in 1890.

As to results obtained, it is stated in the report of 1873 that, even before the dike from the left bank to Colbert Island was begun there was a 2-foot depth in the channel, and to obtain a 3-foot depth at the lowest stage of water it might be necessary not only to construct this dike, but another to connect the "Rock Bar" with the right bank. In the next annual report is the statement that the "Rock Bar" is the most difficult place on the shoals. This report also states that, although the whole amount of work had been completed as originally planned, "the effect expected from it has not been attained because the lack of tightness in the contract-built dams allows the escape through them of the water which they were designed to retain." By 1878 these old dikes had been materially strengthened and tightened, and the district officer's report contained the somewhat too roseate statement that the "general effect has been to increase by several inches the available depth of water, and has reduced this obstruction to one of secondary importance, as indicated by the fact that the pilots now complain of obstructions at other points, whereas, until last season's work was begun this was considered the most formidable obstacle to navigation below Florence. A small amount of work remains to be done, and when completed, this formidable obstruction will be as nearly overcome as practicable without resorting to locks and dams." This small amount of work was done, and the structures were all in good condition when the district officer made his report on the 1887 survey of the shoals. In this report was the following reference to the works of regulation: "The means employed were those usual upon shoals of moderate fall, viz, channel excavation for the purpose of lowering and smoothing the bed of the channel, and narrowing it by spur or wing dams. These efforts seem to have been carried sufficiently far to prove that, when the fall is as great as in this case, a sufficient depth of water can only be gained in this manner by so contracting the channel as to give rise, during low stages of the river, to an insurmountable current. With the improvements now existing, the current is already objectionable, and yet at low water the available depth for several months does not exceed 14 inches.

The report of October 28, 1888, of the board which considered the several proposed plans for the radical improvement of Colbert Shoals contained the following remarks in regard to the previous work done toward improving the navigability of this section: "Work

heretofore carried out for the improvement of Colbert and Bee Tree shoals has consisted in rock excavation and the construction of stone dams for concentrating the water in the channel. This work has possibly resulted in a very slight deepening of the channel at low water, but owing to excessive slope and resulting current at low water and even ordinary stages, which current has been increased by the work carried out, it is apparent that an improvement by this method which would furnish an easily navigated channel at all stages, especially for tows, is impracticable at any reasonable expense. To give a proper depth and a practicable slope and current would necessitate the drawing down of the pools above, which latter result would probably develop new and serious obstructions. All that it appears practicable to gain by continuing this method of improvement will be a slight betterment of the channel for descending navigation."

It must not be inferred, however, from these statements that the work done was not useful, nor that the expenditures were not wisely made in the interests of navigation. During the many years in which the energies of the Department and the appropriations by Congress were being concentrated on the improvement of the impassable Muscle Shoals section, small amounts only were available for the improvement of the channels through Colbert and Bee Tree shoals, and it is very doubtful if the funds at hand could have been expended to better purpose than they were actually expended. No radical improvement was possible with the limited appropriations, and the best that could be done was to attempt to make conditions easier and less dangerous for such navigation as the depths in the channel made possible. Although greater benefit from the works of improvement was anticipated than was actually obtained, nevertheless it may be confidently stated that the work resulted in prolonging the season of navigation and in removing to a large extent the previously existing dangers during the thirty-six years from 1875, when the improvement was reported as completed, until the opening of the lateral canal to navigation on December 14, 1911.

The exact amount expended by the United States on the open-channel improvement of these shoals can not be ascertained, but the total expenditures on the Colbert Shoals section up to June 30, 1890, were $62,343.41, including the cost of the surveys made in connection with studies for the location of a lateral canal. There were at least two such surveys—one of the entire shoals section and the other of the site of the proposed canal—and these must

have cost at least $4,000 to $5,000. In 1893 there was expended $1,903.53 in repairing the dams and in removing a dangerous rock obstruction from the channel. The total amount spent by the United States on the open-channel improvement and its maintenance was, therefore, probably approximately $60,000. No work, except that mentioned for 1893, has been done on the open-channel improvement of the shoals since the adoption in 1890 of the project for a lateral canal. A recent examination by the local United States assistant of the contraction works shows that these stone dikes are still in fairly good condition, and that there has apparently been no deterioration of the channel since the last work was done on the shoals in 1893.

DEVELOPMENT OF THE PRESENT PROJECT.

The Slack-water Plan. In 1887 the district officer in his annual report called attention to the approaching completion of the Muscle Shoals Canal, and recommended that the Colbert Shoals, the principal obstruction below the Muscle Shoals section, should be given consideration, and that the navigable depth for any improvement of the shoals should be fixed at 5 feet at low water, to correspond with the depth provided at the Muscle Shoals section. Following this recommendation, a survey was ordered and was made in 1887. In January of the following year, in a preliminary report, the district officer made a tentative recommendation for the improvement of the shoals by means of canalization. This plan was elaborated in the final report of the district officer, dated February 25, 1888, and provided for two locks and dams located as shown on Plate I; the upper lock being at the foot of Colbert Shoals and the other immediately below the Bee Tree Shoal. It was stated, however, that a site for the upper lock and dam about 2 miles downstream from the position shown would probably be found to be a better site, since it would permit the construction of a longer and lower dam than would be possible at the site first selected. It was proposed to give the lock chambers available dimensions of 60 by 300 feet, and the lifts of the locks were to be 14.5 feet for the upper and 12 feet for the lower. The locks were to be built of masonry, with steel mitering gates, and the dams were designed to be rock-filled timber-crib type, with masonry abutments. The upper dam was to be about 1,700 feet long and 21 feet high, and the lower about 1,350 feet long and 19 feet high. The estimated cost as stated in the report was $923,175. Later examinations,

however, showed that the rock at the lock sites was at much lower elevations than was assumed when this estimate was made, and a considerable addition to this amount would doubtless have been necessary had there been prepared a complete new estimate. In addition to the unsatisfactory foundations at the lock sites, the district officer mentioned the objections to the slackwater plan due to the very high guard required for each of the locks in order to insure that the dams would be "drowned out" at the time of submergence of lock walls and gates—a guard sufficiently great to make the tops of the walls higher than the banks of the river. In the annual report of 1889 the district officer stated that, although no additional survey had been made, a "further analysis of the subject indicates that the estimate then presented will prove sufficient to build the two locks and dams required to overcome the fall upon the two shoals. It is also found that, by putting in longitudinal instead of cross dams, and placing the locks in the side channel of the main river, not only will the cost of the whole be somewhat diminished, but much more satisfactory results will obtain, as this method will leave the main channel unobstructed and available at and above the medium stage. The reduction in cost is due to the fact that owing to the great discharge of the river in comparison to its cross section, fixed cross dams will require lock walls of excessive height to provide uninterrupted navigation during high stages of the river, while longitudinal dams, though necessarily of much greater length, will be of less height, and less expensive construction." A further report by the same district officer in the latter part of 1889 was referred to a board of officers, which made its report in October, 1890. This report contained the statement that "the board has not given detailed consideration to any plan for cross dams and locks and a slackwater improvement, it being in their opinion inexpedient to interfere with the free and unobstructed navigation of the open river, which is practicable during a large portion of the year. If the slackwater improvement of the main channel of the river were decided practicable and advisable its cost would undoubtedly be much less than the cost of the proposed canal." This recommendation caused the laying aside of any further consideration of the proposed slackwater improvement, and work was soon after commenced on a lateral canal. In 1899, however, the slackwater scheme of improvement was again proposed, as will be explained hereafter.

First Plans for a Lateral Canal. The report of the district officer

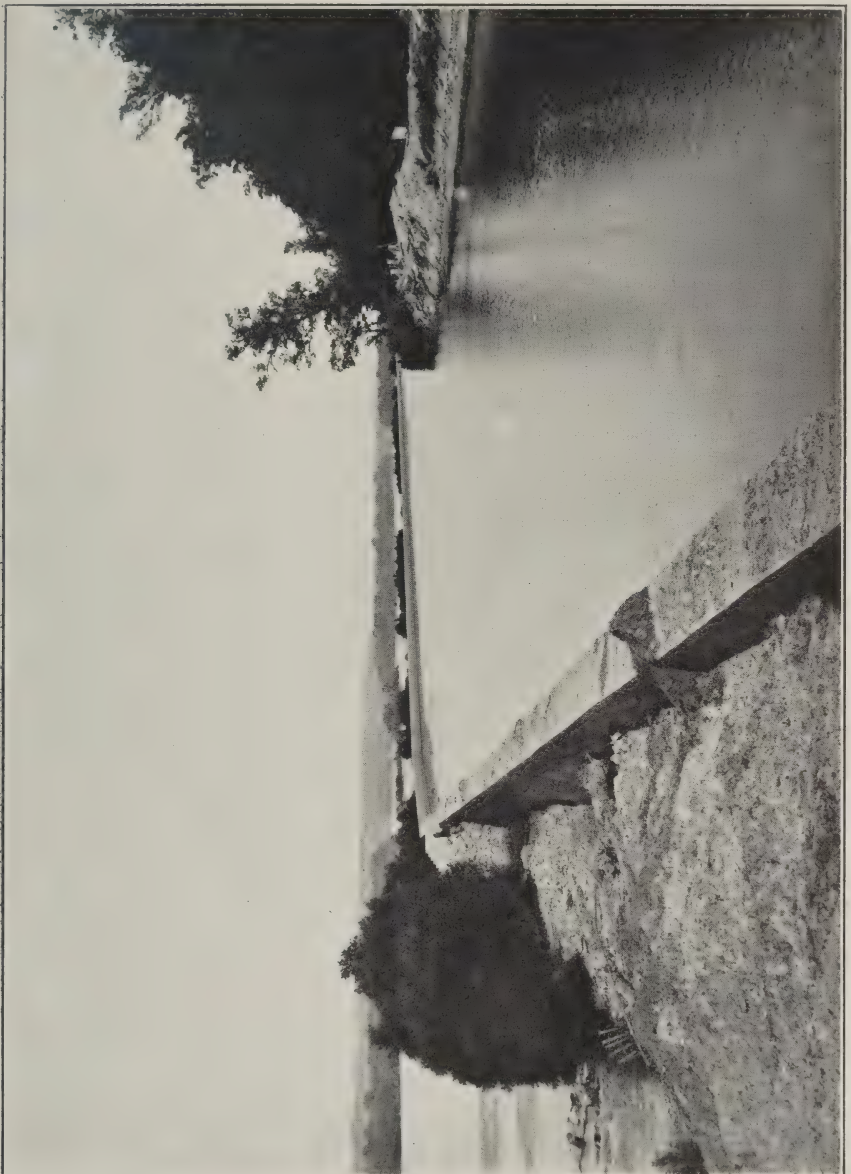


Fig. 5. Lower end of concrete wall, showing Colbert Shoals in the distance.

in December, 1889, contained a description of a plan for the construction of lateral canals in the bed of the river. This plan provided for one canal at the Colbert Shoal, about 3.5 miles long, formed in the back chute by connecting Brush Creek and Colbert islands by an embankment and constructing another embankment running upstream from the head of Colbert Island and parallel to the river bank. The lock was to be constructed near the left bank and at the foot of Brush Creek Island, to which it was to be connected by a rock-filled timber-crib overfall dam. At Bee Tree Shoals there was to be a second canal, 2 miles long, formed in the back chute by connecting Bee Tree and Waterloo islands by a longitudinal embankment 1,200 feet long, and having a lock near the river bank connected to the lower end of Waterloo Island by an overfall dam of rock-filled timber-crib type. The locks were to be of masonry with the same available dimensions as those at the Muscle Shoals Canal, except that there was to be 6-foot depth on the miter sills at low water (instead of 5 feet as at Muscle Shoals). The lock at the Colbert section of the canal was to have a maximum lift of 14.2 feet, and that at the Bee Tree Canal was to have a lift of 11.3 feet. Estimates for the longitudinal walls of the canals were prepared for three types, viz, of masonry, of timber-crib structure and masonry superstructure, and of rock-filled timber-cribs; but the latter type was recommended by the district officer. The plans provided for channel excavation at the head of each section of the canal, and it was proposed to give these excavated channels a width of 200 feet and an ordinary low water depth of 7 feet. The crests of the canal embankments were to be at pool level, except that the 3,600 linear feet of dike just above the Colbert cross dam was to be 2 feet higher, the reason for this exception probably being to prevent a draw over the dam just above the entrance to the lock. The estimated cost of the proposed plan was $836,326.20.

The board which considered this plan made certain important changes in it before comparing it with the plan finally selected for a canal to be excavated in the river bank. The principal modifications suggested by the board were (1) the increase in size of lock chambers to 80 by 350 feet; (2) the increase in height of canal walls and embankments sufficiently to provide still water in the canal until the stage of the river at the head of Colbert Shoals reached 7 feet above low water, after which the canals and locks would be submerged and the boats would use the open river, the

shoals channel then being navigable; (3) the use of masonry and riprap for the longitudinal canal walls, instead of rock-filled timber-cribs (apparently including the utilization of part of the old longitudinal training dike as a canal wall; this old dike strengthened and enlarged probably being the "riprap" part of the canal embankment); (4) the change in the lifts of locks to 14.4 feet for the upper and 9.3 feet for the lower. Plate I shows the plan for two disconnected canals in the bed of the river as modified by the board. The estimated cost, as stated by the board, was $2,750,000; but it was remarked, "in preparing this estimate it has been necessary to take into consideration the difficulties attending the carrying out of excavation and construction in the bed of a rapid river, subject to sudden and frequent rises. Could a constant low water season be depended on the estimate could be materially reduced."

The Adopted Plans for a Lateral Canal. Under the directions of the board there were prepared plans and estimates for a canal to be excavated in the low ground on the left bank of the river. This plan, which was recommended by the board and which subsequently received the approval of the Secretary of War and of Congress, is also shown on Plate I. As described in the board's report, the canal was to be "7.8 miles long and 150 feet wide, excavated through low ground at the foot of the bluffs south of the river and from 100 to 600 feet back of the left bank. Rock cutting is required at the head and foot of the canal. The canal bottom is level throughout and 7 feet below ordinary low water. The banks, for the greater portion of the length, are formed by the natural soil and the excavated material, and are from 40 to 260 feet thick at top. For a distance of about 1 mile the canal passes around the foot of a projecting bluff and occupies a portion of the river bed, requiring an embankment on river bed about 30 feet high. The top of the earthen portion of the embankment is placed 1 foot above the highest authenticated flood, and the embankment in the river bed is above all recent floods. At the lower end of this canal is placed a combined lock with a total lift of 25 feet, and at the head is placed a guard lock for use, when found necessary, at very high water. The banks of this canal are protected by riprap. The canal is made available for use up to the highest stages, and will never be flooded." The locks were to be of masonry, and were to have available dimensions of 80 by 350 feet.

That part of the canal embankment lying within the bed of the river was to consist of two masonry walls 37 feet high, having each

a thickness of 9 feet at bottom and 3.5 feet at top, placed back to back, but separated by a space 3 feet wide, which space was to be filled with clay puddle. This wall would have been overflowed by such floods as that of 1882, and therefore the statement that "the canal will never be flooded" is not correct. The side slopes of the canal were to be 1 on 1½, with a 20-foot berm at about the natural level of the ground. The embankment was to be 1 foot above the 1882 high water, and was to have a minimum top width of 40 feet, with side slopes of 1 on 1½, heavily covered with riprap. The other details of the canal, locks, etc., were not worked out by the board, but apparently were left to be designed by the district officer. The board placed the estimated cost of the canal at $2,500,000.

The board made the following comparison of the relative merits of canals in the bed of the river with the canal in the low ground of the river bank: "The canals in the bed of the river, as compared with the shore canal, furnish, so long as in use, a wider and, for a long portion of length, a deeper channel, and they permit the taking advantage of the stretch of easily navigated river between Colbert and Bee Tree shoals. In other respects the canal on the shore seems to possess advantages over the river works. Its construction will be carried on to a great extent on dry land, rather than in the bed of the river, with a rapid current and subject to sudden rises; the embankments being of greater thickness, with top above flood, and well back from the river, will be more secure and permanent than the walls and embankments of the, at times, submerged river canals; the location of the shore canal near the bluff and behind the high timber along the river bank provides for shelter from heavy winds, which winds would at times interfere materially with the navigation of the excavated channels of 200 feet in the rocky bed of the river canals; and the height projected for locks and embankments provides for the possible use of the shore canal at any and all stages from the lowest to the highest, while in the case of the river canals the use of the open river over a stage above ordinary low water is a necessity. Sediment will be brought into a shore canal from the river through the locks and by the creeks along its route, and this will probably be its most serious defect; but, being easily remedied by dredging, is not considered so great as to outweigh the advantages above considered."

Some of the considerations mentioned by the board as influencing the plans recommended for the improvement of Colbert Shoals are

Fig. 6. Concrete river wall; looking upstream.

of interest, and will be briefly stated. The board very emphatically stated its opinion that the only radical and proper method of overcoming the obstructions at these shoals "is by a system of locks and dams;" and, further, that "the construction of the lateral canals in the bed of the river or on some other feasible route is considered preferable to the construction of cross dams with locks, for the reason that by such plan the main river will be left free and unobstructed" for "about six months, when the shoals can be navigated in their natural state." The importance of this open river navigation seems to have been somewhat magnified by the board. The canal was designed to accommodate a draft of 6 feet, and it was stated that this draft could be carried over the shoals whenever the river stage at the head of the shoals was 7 feet above low water. An examination of the gage records for gage No. 1 indicates that the river is below this stage for an average of eleven months per year. The navigation possible over the shoals for the six months stated by the board can refer only to light draft downstream traffic. The handicap caused for eleven months per year by the restriction of speed in the canal far more than offsets any advantage which the open river channel has over the canal in the short period during which the project draft of 6 feet can pass through the shoals channel.

The board called attention to the fact that the size and draft of boats engaged in "through" business on the river were limited by the size of the Muscle Shoals Canal locks, but expressed the view that "the capacity of the Muscle Shoals Canal should not entirely govern in deciding upon the capacity of the canals or other works of improvement below Florence, there being a large prospective river commerce which will originate in this section of the river and which will not be immediately interested in the river above." This consideration determined the board in deciding on dimensions for the locks of 80 by 350 feet and on an available navigable depth at ordinary low water of 6 feet; whereas, the size of the locks at the Muscle Shoals Canal is 60 by 300 feet and the navigable depth is 5 feet.

In the opinion of the board "the locks and embankments should be given heights of at least 4 feet and 1 foot, respectively, above the water surface of the canal at the highest stages at which the canal is to be used. There should be no current or side drafts in the canal so long as it is used." This statement suggests the idea that the board at first considered a height of embankment which

would permit submersion during flood stages; but, as stated above, the recommended plan provided for sufficient height to bring embankment above the greatest recorded flood except for that part of the canal wall constructed in the river bed, which was to be above all recent floods. Why the board decided to give such heights to the canal embankment and locks is not definitely stated, but it is presumed that it was so done because of assumed danger of breaching embankments if submergible during floods. The quotation from the board's report (see page 630), mentions the fact that the height of locks and embankments provides for the possible use of the canal, however high the river might rise; in fact, this is one of the points of superiority claimed for the shore canal over the canals in the river bed proposed by the district officer. On the other hand, there is some inconsistency in this claim of superiority because of continuous use of the canal, with the great stress laid by the board on the importance of preserving the open river navigation at such times as it would be possible.

The approval of the recommendations of the board on November 28, 1890, was followed by an allotment for the beginning of work on the canal, the allotment being for $150,000 taken from the appropriation of that year for the improvement of the Tennessee River below Chattanooga; and the plans described constituted the first "approved project" for the Colbert Shoals canal.

Earlier Modifications in the Plans. The surveys under way at the time the board made its report were continued, test pits were sunk, and borings were made at the sites of proposed structures; and the canal trunk was staked out, and quantities were calculated. In 1891, following the completion of the surveys, the plans were modified so as to substitute for the double-lift lock at the lower end of the canal two single-lift locks—one at the lower end of the canal and the other, 1 mile therefrom. In 1892, a further modification was made which provided for a single-lift lock at the lower end of the canal with lift equal to the combined lift of the two locks of previous modification. No reasons are given in any of the reports on file for either of these modifications, but the latter was undoubtedly made in the interest of economy, as well as for the purpose of reducing the time of passage through the canal. The changes in estimated cost resulting from these modifications are not stated in the reports, but apparently it was assumed that the changes would not greatly affect the estimated cost of the project.

Contract was let for the lift lock in 1893 under plans providing for a maximum lift of 25.5 feet.

Discussion Following the High Water of 1897, and the Modification of Plans Resulting Therefrom. On August 2, 1899, the district officer submitted a report on the Colbert Shoals Canal showing the necessity for certain changes in the plans, in view of the record high water of 1897. This report is so full, and covers the subject so well, that it is very unfortunate that it was never printed and issued to the Corps of Engineers. At the date of the report the masonry of the lift lock had been completed, the lower entrance thereto excavated, and the guide walls built, and the right-of-way for the canal had been purchased. Detailed plans had been prepared and approved for the guard lock, its gates, and for the upper entrance to the canal. After stating the conditions of the work, the district officer discussed the question of the disposition of flood water from the creeks and brooks whose channels crossed the line of the canal. There were found to be 13 of these streams, draining an aggregate area of 9,500 acres, with a calculated run-off of over 3,000 cubic feet per second. This calculated maximum discharge being beyond the capacity of the culverts of the lock would cause an overflow of the gates of the lift lock, unless it was otherwise disposed of, and would thereby put the canal out of service. There were several possible methods of taking care of the surplus discharge. It might have been allowed to come into the canal, to be taken out over spillways established in the canal embankment with crest lower than the tops of lift lock gates; but this plan would have permitted the flooding of the canal by water from the river passing in over the spillways whenever the surface of the river was above their crests, and thereby would have prevented the continuous usability of the canal as provided for by the original plans. Another plan would have been the increasing of the height of the lift lock gates and the providing of spillways at elevations above extreme high water in the river, and allowing the tributary flood waters either to pass out upstream through the guard locks, with gates opened for the purpose, or over spillways when the height of water in the river at the head of the canal prevented discharge in that direction; but the swift current between the walls of the guard lock would have at times interrupted traffic. A modification of this latter plan might have been used, providing reversed gates for the guard lock, so that the trunk of the canal could fill to the level of the spillways and thus permit the flood water from the tributaries

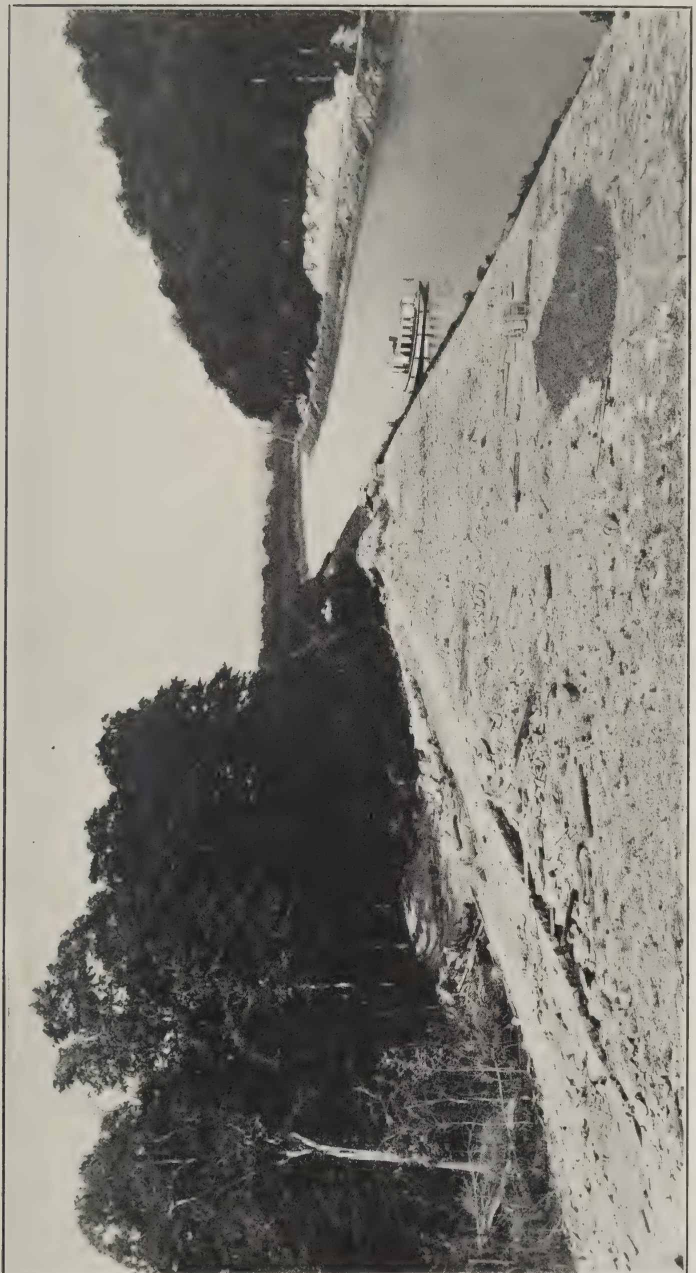


Fig. 7. Waste weir No. 2, from downstream end.

to pass out over the spillways at whatever stage the river might be. The normal level in the canal could have been gradually restored by drawing off the water through the culverts of the locks; but this plan would have been decidedly objectionable, because it would have required an unnecessary lifting of boats by lockage to canal level higher than the river at the head of the canal. It was therefore decided that only a portion of the flood discharge could be allowed to enter the canal trunk. After a careful investigation it was determined that the upper eight streams, being smaller and shorter than the lower five, could safely be permitted to enter the canal, but that the discharge of the other five must be carried underneath the canal by suitable masonry culverts. In designing the culverts it was necessary to prevent the flood waters of the river from backing up through them and thus flooding the canal from the rear, and this was to be accomplished by entirely separating the canal from the valley of the tributary by a levee, with top above extreme high water, passing from high ground on one bank of the tributary to the culvert, along the canal bank, and thence to high ground on the other bank of the creek.

The form of the canal embankment and the exact cross section to be given to the canal trunk were discussed by the district officer, the following being quoted from his report: "The maintenance of the canal seems to depend almost wholly upon the integrity of this embankment. If the embankment should fail at a high stage of water, the canal would undoubtedly be very seriously injured, if not almost completely destroyed. The embankment therefore ought to be as safe and good as it is possible to make it. An earthen embankment when used as a dam may yield, of course, by overturning, but the dimensions which other conditions require generally render this mode of yielding impossible or extremely unlikely; and such embankments sometimes yield by the water finding a passageway through the embankment itself, or through the ground beneath it, and, being forced by a sufficient head, the flow gradually carries out the surrounding material and enlarges the opening until the embankment caves into it and is thus broken and destroyed. Another and perhaps more common form of yielding is due to the sliding down of slopes when they are built at an angle greater than the natural slope of the material when saturated by water. A slope of 1 on 1.5 is very much steeper than any saturated earthy material would assume or maintain. The placing of riprap upon it adds nothing to its stability unless the riprap is sufficient to act as a

retaining wall. When the river is at a high stage, water will be higher upon the outside of the embankment than upon the inside, and if the stage is maintained long enough the embankment will become saturated, and, being unsupported by water on the canal side, this side will slough and slide down until it assumes a natural slope for this condition of saturation. When the river itself falls, if this fall takes place more rapidly than the material of the embankment can drain itself, it is likely to slough and slide down also on the river side. I think, therefore, that a slope such as was proposed is altogether too steep for safety. For the embankment is nearly 8 miles long and more than 25 feet high. I think that it would be unwise to take anything less secure than has been found necessary for good practice in the construction of the levees on the Mississippi River. For this reason, I thought it proper to give to the side slopes of the embankment an angle not less than 1 on 3; and to give it a summit width of 10 feet and to provide a sufficient berm between the toe of the slope and the crest of the side slope of the canal. The next question to be considered was that of the height of the embankment. It was proposed by the board to make it a foot above the flood of 1882. But in the spring of 1897 a freshet occurred which was nearly 2½ feet higher than the highest flood heretofore known. If the embankment had been built in accordance with the plan it would have been overtopped a foot and a half or more throughout its entire length and it could not fail to have suffered very serious injury and probably to have been breached at more than one place, and the material which formed it would have been carried into the canal, partially filling it and rendering it useless, at least, temporarily. Now this flood of 1897 resulted very largely from local rainfall and was not a great flood on the upper part of the river. It would be easy to imagine conditions under which it might have been considerably exceeded, and it seems, therefore, that it would be unsafe to fix the crest of the embankment at only 1 foot above the flood line of 1897. Three feet above the highest known water is usually taken as a factor of safety in the case of levees, and it seems that a no less factor should be assumed in the given case. This will materially increase the height of the proposed embankment, but it seems unavoidable.''

In reference to the double wall of masonry in the bed of the river, the district officer remarked: ''Such a wall will undoubtedly prevent leakage between the river and the canal, and by this construction the canal will take the minimum amount from the channel of

the river. But it would be a very expensive piece of work, and there seems to be no reason why it could not be replaced by an earthen embankment similar to that used along the rest of the canal. * * * It was proposed that this wall be 3 or 4 feet lower than the rest of the embankment; in other words, to have a height such that it would be overtopped by an extraordinary flood. It seems to me that this would be very objectionable, and that the saving that would be effected by reducing the height of the wall would not be worth the danger which would result in the case of a great flood. The slope of the river on the shoals in time of flood is never less than a half a foot to the mile, and if the spillway into the canal a mile long was in existence it is evident that the water surface in the canal at the lower end of the spillway would be practically that in the river, and the fall in the canal would conform to that in the river. As the canal would offer a straight and deep channel the velocity in the canal would undoubtedly be considerable, and would probably be sufficient to produce a dangerous scour at the bottom of the canal in a way to materially injure it in certain places. * * * It seems to me that it is imperative that this wall, whether a masonry structure or an earthen embankment, should be given the same height that is given to the rest of the canal embankment.'' Accordingly, in the estimate it was assumed that the earthen embankment of the canal would be continuous, and of a height throughout of 3 feet above the 1897 flood. The estimate for completion of the canal, with increased height of canal walls and other changes, was $2,749,000, if with earth embankment throughout, and $3,017,000 if with masonry wall in bed of river. Adding these estimated costs for completion to the expenditures to date of the report, or approximately $695,000, the total estimated cost of canal was about $3,444,000 in the first case and about $3,712,000 in the second. These figures were much greater than the originally estimated cost of the canal, and the district officer enumerated the following reasons for the increase: ''First, the actual experience gained in the construction of one lock shows that the board's estimates for locks were too small by $100,000. Second, no estimate was made to provide for safe and easy entrance into the locks. In 1897 a board of Engineer officers was appointed to make plans for an entrance to the lift lock and a plan was submitted and approved; the estimate for it was about $150,000. A similar and less expensive entrance for the upper lock will cost $75,000. These sums were not included in the original estimates. Third,

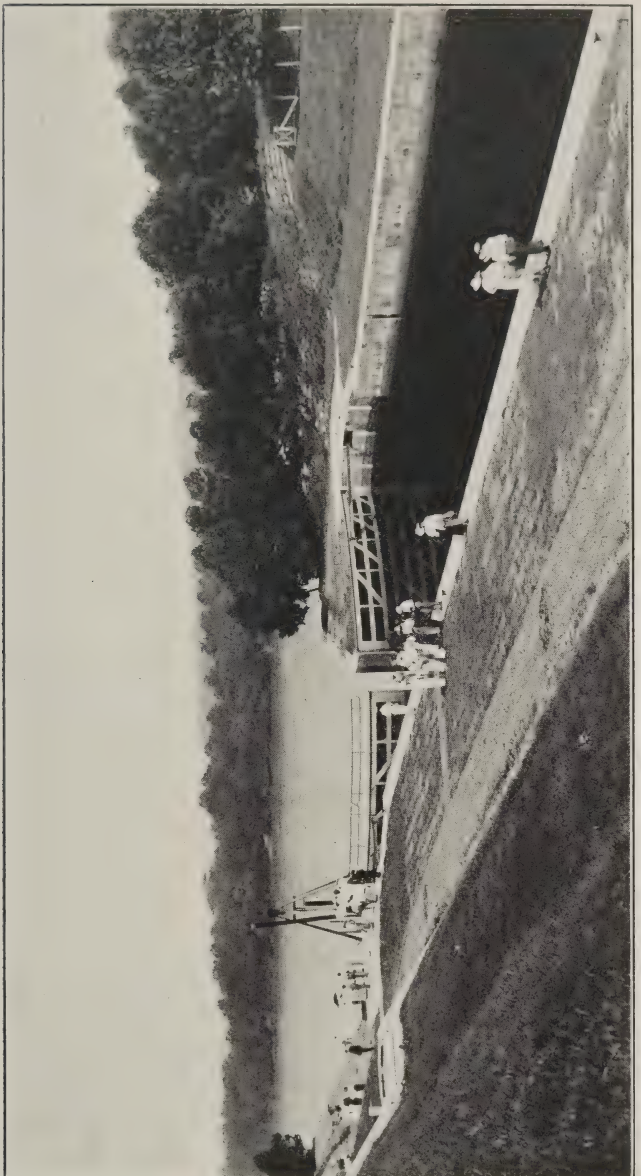


Fig. 8. Lock, from upper end of mound (party of excursionists from Chicago).

the steel lock gates, valves, and operating machinery for the two locks were not provided for, unless they were included in the estimate for masonry, but they are absolutely necessary and will cost $260,000. Fourth, the embankment or levee provided in the original estimate was too low, as heretofore explained. The increased height and the flattened slopes will add $275,000 to the cost of the improvement. Finally, no provision was made in the original project for the disposal of the storm water brought down by the many streams which intersect the line of the proposed canal. The disposal of this water will require extensive and costly works, amounting to $280,000. The sum total of these enumerated amounts is $1,140,000, and this is about the amount by which the present estimate exceeds the one originally prepared.''

In view of this large increase in the estimated cost of the canal, the district officer made a study for the improvement of Colbert Shoals by some method which would cost less than the canal, and as a result of his study he recommended to the Department a plan for the canalization of this section of the river. His plan is designated as the second plan for canalization, and it provided for the use of the lift lock, then completed, with a dam across the river connecting the lift lock with the other bank of the river. Certain modifications were to be made in this lock, and there was to be excavated in the river bank a short canal, or approach, to connect the upper entrance of the lock with the pool formed by the dam. A second dam was to cross the river just above Bee Tree Island, with a masonry lock on the right bank. The lower lock was to have a maximum lift at low water of 9.65 feet and the upper lock of 15.25 feet. The location of the structures included in this plan are shown on Plate III. The estimated cost of the canalization, including the amount already expended, was $2,129,000. Comparing this estimate with that for the lateral canal with earth embankment throughout, it will be seen that the adoption of the plan for canalization would have saved about $1,325,000; but this plan would have provided *only 4 feet depth at extreme low water* in the upper limits of each pool, while the lateral canal would have given a minimum of 6 feet at extreme low water. In the opinion of the district officer, 4 feet at extreme low water was sufficient, in view of the very rare occurrence of extreme low water and of the fact that this depth was all that could be counted on for many years to come over some of the shoals below Riverton. He expected that ultimately additional depth would be obtained at the head of each

pool by rock excavation, but no estimate was included for such excavation.

The district officer's report was referred to a board of officers, which reported that the disadvantages of the slackwater system more than outweigh the probable saving in cost, and recommended that the adopted project for a lateral canal be adhered to, but that the details of the project should be left to the district officer subject to the approval of the Chief of Engineers. The approval of this recommendation definitely and permanently disposed of the question of the improvement of the Colbert Shoals by canalization. While none of the discussions of the board are of record, it is presumed that it based its recommendations largely on the assumed importance of maintaining the open river navigation at high stages.

No detailed consideration appears to have been given by the board to the changes suggested by the district officer in the approved canal plans, but thereafter it seems to have been assumed that the earth embankment was to replace the masonry wall in the bed of the river, and that the embankments and the walls and gates of the guard lock were to be given sufficient height to keep them above such a flood as that of 1897. About two years subsequent to the board's report the district officer described in his annual report the "approved project," showing that it provided for embankments 3 feet above the 1897 high water, with top width of 10 feet, and with side slopes of 1 on 3, which were also to be the side slopes for the excavation. The top of the embankment at the head of the canal was to be at elevation 67.0 and at the lift lock was to be at 63.5. The top of the guard gates was to be at elevation 66.0, and of lift lock gates at 37.0. The canal banks and the guard lock and its gates would therefore never have been submerged, and the canal would have gradually filled over the top of the lift lock gates, which would not be topped until navigation was possible over the shoals. The canal might have been used at even higher stages if desired, by leaving open both upper and lower gates at the lift lock and permitting boats to enter and leave this end of the canal without the necessity for lockage. The maximum pressure against the canal embankment would be outward and at the lower end of the canal, being 25.5 feet at assumed low water; but since the surface of the canal would then be below the natural surface of the ground, the pressure would not act against the made embankment, but against the excavated surface of the natural ground, except where the made embankment crossed the old bed of the tributaries

where extra width would be given to the embankment. Against that part of the embankment lying in the bed of the river the maximum outward pressure would be 12 feet at low water. The maximum inward pressure would be at the head of the canal and just before the water began to come in over top of lift lock gates, the head against the embankment and against the guard lock gates then being 12 feet.

Abandonment of the Plan for a Non-submergible Canal. During the summer of 1901 the district officer and his assistant began a new study of the canal plans with a view toward reducing the cost. On account of the great cost of building embankment and guard lock above the extreme flood level, the question was raised as to what dangers would result from permitting them to be submerged at high water. The question of deposit in the canal was not considered of great importance, since even under the approved plan the water coming in over the lift lock and from the tributary creeks would cause deposit. The district officer did not anticipate trouble from scour along the canal embankments, or within the canal, if built so as to permit submergence at flood stages. He therefore "proposed to limit the embankments and levees at such heights as will insure the canal being available for use until the river attains a stage at which open channel navigation over the shoals is practicable; to provide the embankment on the river side of the canal with open filling weirs, by which, as well as by backing up over the lift lock and by flow through the guard lock, the water in the canal trunk will gradually assume the slope of the river; to discharge into the canal all the thirteen creeks or branches, instead of eight as heretofore approved; and to provide a weir to aid the culverts of the lift lock to discharge surplus waters when the river is low. The reference of the top of the proposed river embankment would range from 42.0 at the waste weir (at lift lock) to 49.0 at the guard lock; the reference of the tops of the main guard gates now proposed is 44.0; the height of the guard gates will be diminished."

It was considered dangerous to permit the canal to fill from the upper end or to be filled by water running over the canal embankment, since the difference in level between the canal surface and the river surface at the beginning of the submergence would be considerable, and there would be a dangerous fall from the river into the canal. In order to avoid these dangers, it was proposed to fill the canal by weirs, or low paved places in the embankment, and to give these weirs such crest heights as would cause them to come

successively into use, beginning with the weir nearest the lift lock. This latter requirement was to be met by "making the grade line for the weir crests to rise faster from foot to head of canal than would the water surface of the river." The establishing of such grade line of weir crests fixed the height of embankment, since at any point the top of the embankment must be not less than 1 foot above the weir; and also fixed the height of guard lock walls, since their tops must be above such grade line. As the weirs must always be submerged successively, beginning with the one at the lift lock, the river slope to be used in determining the grade line of weir crests must be the maximum corresponding to that stage at gage No. 4 which just submerged the lowest of the filling weirs. The elevation of crest for this lowest weir must be at least as high as the river surface at that stage which permitted free navigation in the open river. A careful investigation showed that the shoals channel was *easily* navigable for boats of 6-foot draft, both up and down stream, whenever the river reached a 23.5-foot reading on gage No. 4, elevation 33.5.* Corresponding to such a reading on gage No. 4, *on a rising river* gage No. 1 had reached as a maximum a reading of 10.0 feet, elevation 44.0, and a minimum reading of 7.6 feet, elevation 41.6; while *on a falling river* with maximum slope gage No. 1 had reached 8.3 feet, elevation 42.3, and with minimum slope a reading of 5.0 feet, elevation 39.0. Therefore the maximum recorded fall between gage No. 1 and gage No. 4 for a 23.5 reading on gage No. 4 was 10.5 feet and the minimum was 5.5 feet. When this minimum slope existed there was at least 6.5 feet depth throughout the shoals channel, with moderate currents. When the maximum slope existed the minimum channel depth on the shoals was 11.5 feet, but at that time the currents were swift and a greater margin of depth was required. It was therefore determined that the lowest filling weir could be given crest elevation of 33.5 and that the grade line for weir crests must rise sufficiently to make it above elevation 44.0 at head of canal. Fixing the lowest filling weir at elevation 33.5 would have caused the canal to go out of service when river water began to run in over this weir; but as the lift lock had already been completed with top of walls at elevation 37.0, it was apparent that the canal could be continued in service up to a

*On page 616 it may be noted that 6-foot draft can be moved upstream when Riverton gage reads 17.5 feet, corresponding to a reading on gage No. 4 of 19.5 feet. The assumption of a gage No. 4 reading of 23.5 feet for *easy* navigation was clearly on the safe side.

gage No. 4 reading of 27.0, elevation 37.0, if the lowest filling weir was given crest elevation of 37.0. However, it was decided to continue use of canal only to the 26.0 stage, and therefore the lowest filling weir was given crest elevation of 36.0.

While, as has been stated, the grade for weir crests was to be steeper than the maximum slope in river corresponding to the stage which first submerged the lowest of the weirs, the question of how much steeper was still to be decided. Two things were considered in fixing the minimum rise in grade of weir crests—provision against the possibility that the maximum recorded slope in the river might be exceeded in the future, and an allowance of time for filling the trunk of canal to keep pace with the submergence of the weirs. For this latter reason the grade line for weir crests should be steeper near the lower end of the canal than at the head, since the length of trunk to be filled decreases as the submergence proceeds upstream. Consistent with these two requirements, the grade line should be fixed on as flat a slope as practicable in order to reduce the resulting heights for embankment and for guard lock walls. Unfortunately an error was introduced in the plans by the use of the maximum slope for the river corresponding to gage No. 4 reading of 23.5 feet instead of 26.0 feet, which had been fixed on as the closing stage for the canal. The maximum fall in the river corresponding to the 26.0-foot stage was 9.5 feet instead of 10.5 feet, as assumed, and this error caused the weir crest line to rise somewhat faster than necessary. No danger was introduced thereby, but the embankment at the upper end would have been 1 foot higher than required, with corresponding additional cost. Other considerations further modified the height of embankment. The weir nearest the guard lock was 7,000 feet therefrom, with crest at 48.0 and with top of embankment at 49.0. From this point to the guard lock the embankment gradually rose to elevation 50.0, but the rise must have been somewhat greater had the grade line of weir crests been carried as far as the guard lock. Below this uppermost weir the embankment was to be 1 foot above the crest line of weirs, except where excess of excavated material to be wasted would have given embankment higher than required by this rule.

It was proposed to take the storm water discharge of all thirteen streams directly into the canal, and provision was therefore necessary for taking care of this water either by discharging it over waste weirs or through culverts. There were numerous opportunities for failure of operation of the latter, with consequent danger

of overtopping the lift lock gates and thereby putting the canal out of service. A waste weir was therefore provided at the lift lock, discharging into the old channel of Chickasaw Branch. This weir was to be a concrete structure founded on rock, with crest 33 feet long and at elevation 33.5, permitting a depth of discharge over the crest of 2.0 to 2.5 feet without the water in canal rising above 35.5 or 36.0, or to within 1½ or 1 foot of the tops of the lift lock gates. Calculations showed that the maximum current in the canal due to storm water could never exceed 2.5 feet per second, and probably would not be more than 1.8 feet per second. Even 2.5 feet per second would not injure the paved slopes of the canal, and would not greatly interfere with navigation. The waste weir was to have a culvert at level of bottom of canal to aid the lock culverts in emptying the canal whenever that became necessary. The waste weir being at elevation 33.5 would allow the canal to begin filling when the river surface at lift lock was above this elevation. The small amount of such filling over the short weir was not expected to interfere with the use of the canal until a further rise topped either the lift lock gates, or the guard lock gates, or the lowest of the filling weirs.

Since the natural level of the ground along the lower half of the canal was higher than the grade line for weir crests, it was necessary, in order to give the weirs free connection with the river, to locate them only at the intersection of the canal embankment with the old creek beds. The weirs were to have sufficient widths to insure easy filling of the canal trunk as the river rose, and were to be simply paved spaces across top of the embankment; the pavement being of 1-foot concrete blocks and extending across top and down the canal slope to elevation 33.5.

The embankment was to have a minimum top width of 40 feet, with side slopes of 1 on 3. On the river side, that part of the embankment in the river bed, as well as about 1,700 linear feet situated near the river bank, was to be covered from bottom to top with heavy riprap of an average thickness of 1 foot. The canal side of the embankment throughout was to be covered with lighter riprap, 1 foot average thickness, from bottom of canal (elevation 26.5) to 2.5 feet above assumed low water, or to elevation 36.0. That part of the embankment in the river bed, which would be on bed rock, was to have a 6 by 6 foot concrete core wall to insure a good bond between the earth embankment and the underlying rock. The walls of the guard lock were to be built to elevation 49.0, although the

tops of the gates were to be at 44.0. No reasons are given for this extra 5 feet height of walls, but it is not unlikely that it was with a view to an increase in height of gates if later it was found to be necessary.*

It will thus be seen that the proposed modifications reduced the heights of guard lock walls and of canal embankment by 17 feet and eliminated the expensive works (levees and culverts) for carrying the storm water of the five larger creeks underneath the canal trunk. On the other hand, the original plans provided for embankment and guard lock to be above highest floods, for no current through the canal, and for its being usable at all river stages; while the modified plans permitted the embankment and the guard lock to be submerged, a current in the canal (although not a dangerous one), and the canal going out of service when gage No. 4 was at or above 26.0. (It was stated by the district officer that the gage records for twenty-nine years showed that the canal would have been out of service on an average of only five days per year.) The estimate for completion of the canal in accordance with these plans including $788,000 previously expended, was about $2,500,000; and in accordance with the ''approved project'' was $3,444,000. The modifications apparently gave a canal as serviceable as under the former plans, and one not more subject to accidents or damage, and at an estimated saving of about $944,000. The proposed modifications having been approved by the Secretary of War, the ''Approved Project'' was modified accordingly.

With a canal constructed according to the new plans, the conditions for the filling of the canal would vary considerably under different river conditions. The following is a brief explanation: (a) *Rising river and maximum slope.* As river at lift lock reached 33.5, at guard lock it would reach 44.0; and the next increment of rise would cause the canal to begin filling over the waste weir at the lift lock, and over the guard lock gates. A further rise to 37.0 at the lift lock would top the gates, and at about the same time the lowest filling weir (at Bledsoes Creek) would begin to flow; the other filling weirs would successively go under—the uppermost at Boones Branch shortly before the flooding of the guard lock walls, which would be at 15.0 feet on gage No. 1. The embankment would

*About one year later the plans for the guard lock were further changed to provide for tops of walls at elevation 50.0 and of gates at 46.0. This change, however, is not considered in the next few paragraphs, as the plans for the guard lock very soon afterwards were still further changed.

be entirely flooded (except where wasted excavation made it unnecessarily high) at a 16-foot stage at gage 1, elevation 50.0. (b) *Rising river and minimum slope.* When the river at lift lock reached elevation 33.5, at guard lock it would be at elevation 41.6. A further rise would cause canal to begin filling over the waste weir, but the canal would not go out of service until a further rise to elevation 37.0 at the lift lock had flooded the gates, at which time the river at guard lock would be at elevation about 43.2. Then there would be flooded the lowest filling weir, and possibly the next, before the guard lock gates would be topped. The order of submergence would thereafter be as given in (a). (c) *Falling river and maximum slope.* As the river at guard lock fell to elevation 50.0, the upper end of embankment would be uncovered, followed progressively by remainder of embankment. In order then would appear the tops of guard lock walls, the Boones Branch weir, and the other weirs in succession; then the lift lock gates and guard lock gates. When the river had fallen to 44.0 at guard lock, it would be at about 35.4 at lift lock. The waste weir would appear when river had elevations 33.5 at lift lock and 42.3 at guard lock. (d) *Falling river and minimum slope.* The embankment, the guard lock walls, and the filling weirs would appear in the order stated for (c). When the river at the guard lock reached 44.0, at the lift lock it would be at about 37.1 or 37.2; that is, guard lock gates would appear before lift lock gates. Soon after the canal would go into service and a further fall would bring the waste weir above the river surface.

Change in Location of Guard Lock, and Location of the Upper Part of the Canal Trunk in the River Bed. The same district officer, after further study of the probable discharges of the different tributary creeks, concluded that unless waste weirs were provided at frequent intervals in the canal embankment there would be great likelihood that velocities in the canal due to storm water would be so great at times as to scour the bottom and sides of the canal. During the study for the location of such waste weirs he further concluded that there was no good reason for the high embankment nor for the guard lock. He therefore proposed to the Chief of Engineers in September, 1902, to eliminate the guard lock and all the embankment on the river side of the canal, except where the canal was not entirely in cut; that is, except where the canal embankment lay in the river bed or where it crossed one of the tributary streams. This proposal will not be described in detail, as

it was not approved, and as it is discussed to some extent in connection with a change of plan proposed at a later date. A board was assembled to consider and report on this proposed radical change of plan, and it expressed the opinion that the guard lock should not be eliminated; but a suggestion was made to the Chief of Engineers that a number of different plans for the canal be studied and estimates therefor be prepared. In pursuance of this suggestion the district officer submitted estimates for each of the plans outlined by the board, and in addition he proposed a new plan, a modification of one of those of the board, prepared estimates for it, and recommended it to be adopted.

In this plan the location of the guard lock was changed to a point about 150 feet above Shades Branch; the canal between the two locks was to be altogether in cut and to be as near the river as the right-of-way and the side slope of the canal would permit. The bulk of the excavated material was to be deposited on the land side of the canal; and, in order to reduce excavation, the side slopes of the cut were to be made 1 on 2, leaving the bottom width, as before at 108 feet, but reducing the top width; and furthermore it was proposed to pave the side slopes from bottom to top. Upstream from the guard lock the canal trunk was to be formed by constructing a concrete wall in the river bed paralleling the river bank, and by excavating rock where necessary to give proper depth. The bottom width of this part of the canal was also to be 108 feet; the rock cut having vertical walls. The concrete wall was to be 4 feet wide for the upper 3¾ feet, and below that was to have a vertical face on the canal side and 3 on 1 batter on river face. The lower 5,250 feet of the wall was to have its crest at elevation 41.0, to serve as a spillway, while the remainder was to have top elevation at 42.0. The tops of the guard lock walls and gates were fixed at elevation 43.0, as was also the top of the earth embankment where it had to be built up. Elevation of miter sills of guard lock was to be at 27.5. There were to be five waste weirs, elevation of crest of each at 34.0, located at Chickasaw, Bledsoe, Hayes, Ross, and Boones branches. These were for the purpose of discharging the storm water of the tributaries, as well as for filling the canal as the river rose. The estimate for completion was $2,183,235, including previous expenditures, as against $2,853,608, estimated total cost for completing the canal according to previous project. The proposed plan having been approved by the Secretary of War, it became the "approved project;" but not very long afterwards a

further modification in the interest of economy provided for the substitution of a rock-fill embankment for the concrete wall in the upper 5,500 linear feet. This embankment was to be built of the rock excavated from the canal trunk, piled on and around the cofferdam used for unwatering the river bed for the purpose of rock excavation, and was to have a low concrete toe wall 2 feet thick extending to elevation 34.0, above which the rock slope was to continue to elevation 42.0, the proposed top elevation. This substitution was to extend downstream only so far as the amount of rock to be excavated was sufficient to give a minimum thickness of 40 feet to the rock embankment. The rock surface sloping gently downstream, the excavated material would be sufficient to give this minimum thickness for only the upper 5,500 linear feet. The concrete wall was for the purpose of insuring tightness below the low water surface of the canal, and above that surface some leakage through the rock embankment would not be objectionable, although it was expected that deposit from the river would soon consolidate the embankment. This change was estimated to make a saving of $50,000.

(*To be continued in January number.*)

A Japanese Winter Exercise*

BY

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The winter exercises held by the 16th Division between the 12th and 16th February were designed to include the following events: On the first day an encounter battle on the Tamamizu-Tanabe-Yawata road; on the second day, an attack by the Red Force on a Blue position in the hills; on the third day, retirement of Red across a river pursued by Blue; and on the fourth day, the active defence of the line of the river by Red. (See Fig. 1, page 653.)

The situation on which the exercise was based was as follows:

GENERAL IDEA.

A Blue army is holding the line Yokoogi-Mukomachi.

A Red army is advancing via Fushimi against Blue.

Special Idea. Blue.

Morning of the 12th February.

The Blue commander has decided to retire to the heights of Otokoyama and Yamazaki and await the arrival of reinforcements before accepting battle.

Upon receiving information that a small force of the enemy is advancing down the Kizu Valley, the Blue commander detaches the force detailed below to protect the rear of his army:

1st and 2d Battalions of the 9th Infantry Regiment; 2d Squadron of the 20th Cavalry Regiment; 2d Battery of the 22d F. A.

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*Reprinted, by permission, from the *Journal of the Royal United Service Institution*, London, January, 1913.

Note. The translation (from the *Militär Wochenblatt* Supplement No. 1 of 1912) illustrates admirably various tactical ideas prevailing in Japan as the result of the experience gained in the war against Russia.

It likewise shows what valuable instruction can be acquired by means of exercises with small forces and partly improvised formations; and despite inclement weather conditions similar to those which usually prevail during the "winter training season" in Great Britain.

The criticisms by the author—an expert foreign observer—constitute a valuable addition to the instructive events which he narrates.—Ed.

Regiment; and two sections of the 2d Company of the 16th Pioneer Battalion.

This force, which is henceforward referred to as the Kizu detachment, had reached the bridge south of Yodo at 12 noon. Its commander had received information that the Red Force, which had hitherto been advancing on the right (east) bank of the Kizu, had reached the neighborhood of Tamamizu, and was preparing to cross to the left (west) bank. The inhabitants reported that the Yawata-Matsui road was passable for field artillery.

Special Idea. Red.

The Red Kizu Detachment (as below) had received orders to operate against the rear of the Blue army; it had advanced on the east bank of the River Kizu as far as Tamamizu, where it had crossed to the west bank on the 12th February. The crossing had been completed by noon. The Asasano-Matsui-Yawata road was reported to be passable for field artillery.

1st and 2d Battalions of the 53d Infantry Regiment.
Half the 1st Squadron of the 20th Cavalry Regiment.
1st Battery of the 22d F. A. Regiment.
One section of the 2d Company of the 16th Pioneer Battalion.

The total forces engaged in the exercise were, therefore, quite small. There were altogther two regimental staffs, four infantry battalions, two cavalry squadrons, two batteries of field artillery, one company of pioneers and eight machine-guns; second-line transport was represented by detachments of A. S. C.; all units were at war strength,* with their full allowance of pack ammunition.

The Directing Staff consisted of General Oi, commanding the 19th Infantry Brigade, and his two staff officers; in addition the 16th Division was given its "I b"† General Staff Officer. My Austrian friend, Captain von Winternitz, and I were attached to the 16th Division. On the 12th February the Directing Staff left Kioto for the maneuver area.

All officers who were not absolutely required for training the recruits in the garrisons took part in the exercise, and were employed as umpires, compensation officers, to replace casualties among the commanders, or as spectators.

The weather was slightly frosty; the sky was generally clear and the wind keen; these conditions continued almost till the end of the exercise, but at noon on the 15th there was a blizzard, which was followed by a change of the weather.

The four days were utilized to the very utmost, having regard to the limits imposed by the physical endurance of the troops; the

*In order to bring units up to war strength some reservists were called up to each regiment from the place at which it was stationed.

†The expression "I b" is used in the German Army to designate one of the two officers of the 1st (General Staff) Section of an Army Corps Staff.—Ed.

maneuver area was comparatively small, but it was well selected and afforded plenty of scope for interesting situations, which were exploited to the utmost by the maneuver scheme. The result was to give the older men of the division and the majority of the officers a thorough "Refresher," in the middle of the winter, without interfering appreciably with the training of the recruits.

In order to reach the places of assembly laid down in the scheme, the infantry had to proceed by route march 12½ miles from Fushimi, 9 miles from Nara and 12½ miles from Ōtsu. This is in accordance with the approved Japanese tradition of taking the edge off the marching powers of the troops before the maneuvers begin, in order to test their energy and capacity for endurance.*

At 12.30 p. m., by which time the concentration had been completed, the troops had rested, and were ready to move off. The situation was then as portrayed by the following order:

RED. DETACHMENT ORDERS.†

Tamamizu, 12th Feb., 12 noon.

Order of March.

Independent Cavalry.—1st Squadron 20th Cavalry Regiment.

Advanced Guard.—1st Battalion of 53d Infantry Regiment (less Nos. 3 and 4 companies); four mounted men; one section 16th Pioneers.

Left Flank Guard.—2d Battalion 53d Regiment (less No. 2 company); two machine guns.

Main Body.—Nos. 3 and 4 companies 1st Battalion 53d Regiment; 1st Battery 22d F. A. Regiment; No. 2 company 2d Battalion 53d Regiment.

1. No news of enemy.

2. The detachment will march to Yawata, towards which place the independent cavalry advanced at 10.30 a. m. this morning to reconnoiter.

3. The infantry point of the advanced guard will move off at 12.30 p. m. along the Tanabe-Yawata road upon Yawata.

4. The left flank guard will move off at the same time as the advanced guard and proceed from Tanabe via Matsui and Uchisato to Yawata.

5. The main body will move off at 12.30 p.m. from the center of Tamamizu village.

6. Second-line transport will follow the main body at 1,100 yards distance.

7. The G. O. C. will ride at the head of the main body.

(Note: No infantry may cross the railway north of Tanabe before 2 p. m.)

Dictated to commanding officers at Tamamizu.

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*Thus, for example, the inspection of companies in tactical training is usually arranged as an incident in a 25-mile march.

†For Director's criticism on these orders, see pages 666-667.—Ed.

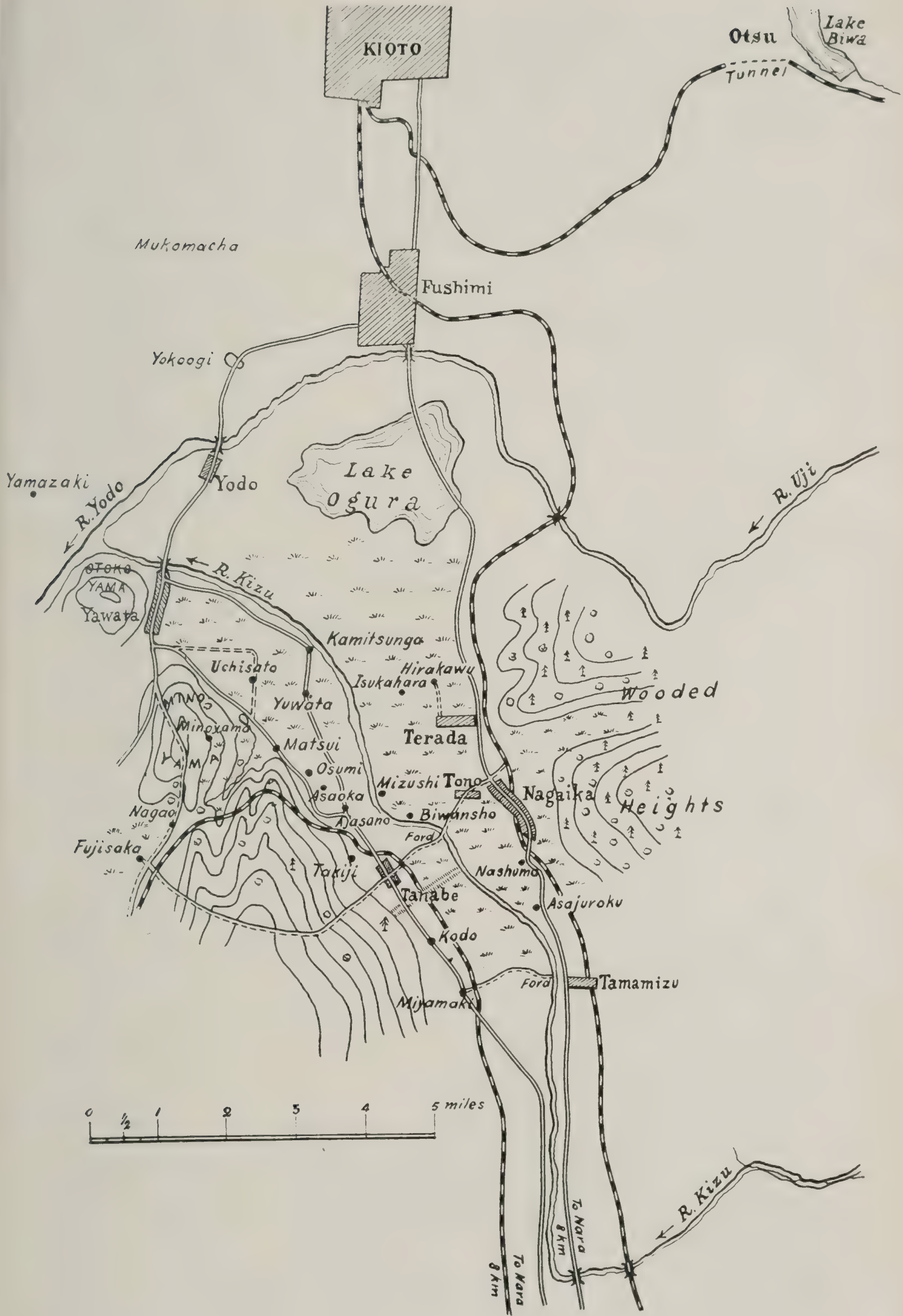
KIOTO
Otsu
Lake Biwa
Tunnel
Mukomacha
Fushimi
Yokoogi
Lake Ogura
Yamazaki
R. Yodo
Yodo
R. Uji
R. Kizu
OTOKO YAMA
Yawata
Kamitsunga
Uchisato
Hirakawu
Isukahara
Yuwata
Wooded
Minoyama
Matsui
Terada
Osumi
Mizushi
Tono
Nagaika
Heights
Asaoka
Nagao
Biwansho
Fujisaka
Ford
Takiji
Nashumo
Tanabe
Asajuroku
Kodo
Ford
Tamamizu
Miyamaki
0 ½ 1 2 3 4 5 miles
To Nara 8 km
To Nara 8 km
R. Kizu

Fig. 1.

The Blue commander had likewise resolved to advance. His orders were that the advanced guard should start at 12.30 for Tanabe via Yawata, the main body following at a distance of 770 yards. On reaching Yawata a flank guard consisting of two companies was to diverge from the main body and move via Matsui on Takigi.

NARRATIVE OF THE OPERATIONS.

The Independent Cavalry of the opposing sides, having been allowed to move off at 10.30 a. m., came in contact between Yawata and Osumi.

The Red advanced guard, which I accompanied at the commencement of the day's operations, sent out some infantry patrols at the double on a broad front to prevent the hostile cavalry from approaching within view. A little after 2 p. m., the point of the Red advanced guard came in contact with the Blue cavalry on the narrow plateau between 200 and 500 yards south of Asasano; the Red advanced guard hastened to deploy, and occupied a trench extending on either side of the road, 500 yards south of Yawata; from this position it was able to open an effective fire against the Blue advanced guard, which was debouching somewhat carelessly from Yawata.

Some weak flank guard detachments and patrols were engaged a little farther to the east, in an independent struggle for the river dike, which is as much as 30 feet high in some places.

The road leading through Yawata was in such bad condition that some pioneer work was necessary before the Blue guns could get into action, which they finally did under fire of the guns and advanced guard of Red, in a position on the southwest border of Yawata (which, in real war, not a man would have reached alive). An encounter battle now commenced.

The energetic Blue commander quickly deployed his main body and sent it forward through the fields southwest of Yawata; meanwhile Red advanced by his left between Osumi and Matsui; the infantry, moving in close formation, sank deep in the marshy rice fields, and at the same time came under fire from the Blue machine guns.

Great effect is allowed in Japan to surprise fire by machine guns from a flank, on account of the general confusion which fire of this sort was found to produce during the late war.

The position of Red at 3.30 p. m. when "Stand Fast" was sounded, was by no means favorable, although they had established a strong firing line along the embankment on which the road is carried to the east of Matsui.

At 5 p. m., after a long conference, operations were resumed. Red retired via Tanabe; Blue pursued half-heartedly and allowed themselves to be checked by fire at the embankment 200 yards south of Asasano; they halted again at the railway north of Tanabe, and at a place where an embankment is carried over the road at the

latter place. The greatest eagerness in the pursuit was shown by the Blue battery commander with his mounted party; the machine guns followed him at a rather slower pace. It was not until the evening that Blue halted at the embankment south of Tanabe in accordance with orders received from Army Headquarters.

Red had fallen back to Kodo, where they were turning to face their enemy, when the latter desisted from the pursuit. The Red commander had just given orders for his force to occupy close billets, in immediate readiness for action, when he received information from his army commander that he was going to begin the attack on the Blue Army on the 13th, on which day his left flank would move against the Mino Hill south of Yawata; and that the task for the Kizu Detachment was to drive back the Blue troops opposed to them, and then to cooperate on the extreme left flank of the Red Army's attack.

It was at first proposed, in pursuance of this order, to gain possession of Tanabe by a surprise attack in the darkness, but the patrols, which had been gradually working their way round the the enemy's thin screen of posts, reported that neither Tanabe nor Takigi were occupied, and that the Blue outpost line was really much farther to the north, south of Asaoka and Asasano. The Blue detachment had received news after the action, from the 16th Division (on the right flank of the Blue Army) that it was falling back that very night to take up a defensive position south of Yawata. Shortly afterwards the following extract from Blue Army Orders was received, dated 12.30 p. m.:

"1. On the 13th inst., the 16th Division will occupy the line from (right flank) hill 880 yards south of Minoyama—Village of Minoyama—Hill Otokoyama; on the 14th this division will again advance to the attack.

"2. The Kizu Detachment will to-night occupy a position to cover the retirement of our main army; early to-morrow it will move to join the right wing of the 16th Division and will entrench itself there, facing east."

The effect of these orders was that Red was only followed up by a portion of the Blue Force as far as the dyke south of Tanabe, where a false outpost line, which deceived Red for a time, was taken up by cavalry and patrols. Judging rightly that the first task was to gain time to fall back towards the northwest, the Blue commander now issued the following orders:

BLUE. DETACHMENT ORDERS.

Asaoka, 12th Feb., 6.30 p. m.

1. The enemy has been driven back through Tanabe, and has taken up quarters for the night at Kodo.
2. Our detachment is quartered to-night in and around Asaoka.
3. The 1st Battalion 9th Infantry Regiment will furnish the

outposts; reserve will be at Asasano; right flank will extend to the tea plantation 770 yards south of Asaoka and the western slope of the hill at that place; left flank will be at the ford by the gap in the dyke opposite Mizushi, and will guard by frequent patrolling against any movement from the direction of Tanabe.

4. The main body will occupy close billets as under:

Staff, machine guns, 2d Battalion of 9th Regiment, less No. 2 company at Asaoka; No. 2 company 9th Regiment at East Ozumi; No. 2 squadron 20th Cavalry Regiment and No. 2 company 16th Pioneers at West Ozumi; 2d Battery 22d F. A. Regiment at Matsui.

Major T. will be commandant of Asaoka.

5. The alarm post will be in the fields southeast of Asacka.

6. Rations will be issued from the wagons of the second-line transport (*Grosse Baggage*).

7. I shall remain at Asaoka, where orders will be issued at 9 p. m.

By 9 p. m. the troops on both sides had received their evening issue of supplies (or the amount required to complete it) and had turned in under the protection of their outposts, who were kept busy all night.

The commander of the Red Kizu Detachment issued the following orders for the 13th February:

RED. ORDERS FOR THE KIZUGAWA DETACHMENT.

Headquarters, Kodo, 12th February, 9 p. m.

1. The foremost protective detachments of the enemy opposed to us are on the line of the high river bank south of Asasano-Asaoka. The main body of the enemy has fallen back and apparently intends to resist our advance in the hills at, and to the north of, Minoyama.

The Kizugawa Detachment will to-morrow advance further along the left bank of the Kizu.

2. To-morrow (13th February), the enemy will accordingly be attacked afresh; after defeating him, the detachment will establish touch with the Red main body.

3. The 2d Battalion 53d Regiment, less two companies, will occupy the railway embankment north of Tanabe at 5 a. m. to-morrow; the machine guns will accompany this battalion.

4. At the same hour the 1st Battalion 53d Regiment and two companies of the 2d Battalion 53d Regiment, under the command of Major F., will be assembled north of Tanabe.

5. At 5.30 a. m. the battery will be "at the north of Tanabe" (by this was meant in action at the northern end of the village): one section of pioneers will accompany the battery.

6. The cavalry squadron will endeavor to reconnoiter along the left bank of the Kizu, and will cover our right flank.

7. Communication is to be established from Tanabe via Nagaike to the road leading from Nara.

8. Second-line transport will remain till midday to-morrow on the

right bank of the Kizu near Tono, to be halted in order of units as given in the order of battle.

9. I shall be at the cross roads in the center of Tanabe at 5.30 a. m.*

The intentions of Blue were also modified in many respects. The original intention had been to move off quickly to the northwest an hour before daybreak, and then to take up a position for a delaying action in the neighborhood Asaoka-Osumi, the rear guard remain-

Fig. 2.

ing south of Asasano; finally, it was decided to fall back on the heights 200 yards southeast of Minoyama, and to hold on there, linking up to the 16th Division. With this object, orders were issued at 2 a. m. on the 13th February that the battery was to fall back through Matsui and was to be formed up at 5 a. m. on the hill 880 yards northeast of Nagao; the 2d Battalion 9th Infantry Regiment was to be at 5.20 a. m. at the pond 500 yards northeast of Nagao; and the 1st Battalion 9th Infantry Regiment was to be at

*Unfortunately, by order of the Directing Staff, all operations were postponed till two hours later than the times prescribed in the orders.

5.40 a. m. at the ponds 660 yards south of Minoyama (see Fig. 2). One company of each battalion was to be left behind to mask the retirement and to afford some support to the cavalry which was going out to reconnoiter.

Blue carried out his retirement on to the selected position with great skill and succeeded in completely hoodwinking the enemy;* the heights south of Matsui were found to provide good positions for rear guard actions.

The Red commander unaware of the retirement of Blue, kept four of his guns in action to the west and two to the east of Tanabe; his remaining troops he kept in the positions laid down by the orders, up to 7 a. m., without having been able to decide on any definite action. He then advanced astride of the road, and was enticed by the Blue rear guard, which played the rôle of a delaying force with consummate skill, into the country southwest of Matsui. Here all touch was lost with the Blue troops, who vanished suddenly and completely. At this juncture, Red received information that there was a strong Blue position northwest of Minoyama; and, further, that the advanced troops of the Red main body (which was now moving to the attack) would not appear north of Matsui before 4.30 p. m. The Red commander realized, therefore, that he must proceed with caution in regaining touch with the enemy who had so mysteriously disappeared; meanwhile, he took steps to secure his position at Matsui and on the ridge south of that place, facing towards the west. (See Fig. 2.)

In the meanwhile, we spectators had made a detour, under the guidance of a General Staff Officer, and had reached the new defensive position of the Blue detachment. The ground, which was the scene of the ensuing operations, presented all the difficulties of mountainous country, though the actual altitudes above sea level were relatively small; it bore a striking resemblance, as was more than once observed by the Japanese officers, to the battlefield of Yang-tzu-ling in the Manchurian mountains.

During our ride we saw the Red advanced guard regain touch with the enemy; in so doing it was on one occasion much exposed to fire from the strong Blue position. The Red infantry patrols were soon busily engaged, and worked well. The heavy ground presented great difficulties to the advance of the guns; the artillery reconnoitering parties selected a position on the hill 550 yards southwest of Matsui, but before it could be occupied the pioneers had to make a track over 200 yards of difficult ground; when this had been done the position was certainly an excellent one for artillery.

The Blue defensive line, which we reached about noon, by way of a steep narrow valley, was supposed to constitute the extreme right of the division on the right flank of the Blue Army. The front, which was about 1,540 yards in extent, was only occupied, in the

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*See Director's criticisms on pages 666-667.—Ed.

first instance, by one battalion of infantry, four machine guns, and one battery; the latter was employed in dispersed sections, but was kept under one control.

There was a double system of telephone communication between the various command posts and intelligence collecting stations (*Meldestellen*); and also to the advanced posts, to the commander, and to the reserve. The strength of the last named was one battalion; it was at first judiciously stationed in rear of the right flank, but later, in view of the enemy's preparations for a strong frontal attack, it was moved to a position in rear of the center.

Blues were allowed plenty of time to strengthen their position; they were thus able to dig deep trenches, and to construct command posts, gun emplacements, covered communications, screens and dummy trenches; wire entanglements and abattis of pointed bamboos (the latter as dangerous in peace as in war) were also constructed on a large scale. By 3.30 p. m. the defensive position had become a perfect specimen of the art of field fortification; the work had been well executed and the cover provided was good and substantial. Nevertheless, taken as a whole, the position suffered from the defect that the flank was too far advanced. In spite, therefore, of all the local strength afforded by the wire entanglements and pointed bamboo stakes the flank was unduly exposed to an enveloping movement.

The effects of the orders issued by the Red commander for the further operations that afternoon began to be visible about 3.20 p. m. These orders were as follows:

RED. ORDERS FOR THE KIZU DETACHMENT.

Hill 1 kilometer south of Matsui, 13th February, 2 p. m.

1. The enemy is occupying a strong position on the Matsui-Nagao road extending for about 440 yards on either side of the column. Some reserves are in the neighborhood of the pond 550 yards north of Nagao. The positions of his defences and outposts have not yet been precisely determined.

2. To-day the detachment will maintain its present position, in rear of which the troops will bivouac. We will await the advance of our main army (which is to engage the enemy to-morrow) and then attack at daybreak to-morrow.

3. The 2d Battalion 53d Regiment (less one company) will advance on both sides of the valley and make good the ridge.

4. The 1st Battalion 53d Regiment will hold the line from the mountain with the pinewood as far as the railway culvert near the tunnel.

5. The cavalry will advance towards Matsui in the evening and will establish touch with our main army, which is advancing north-east of that place.

6. The 1st Battalion 53d Regiment will protect the artillery position and bivouac.

7. The section of Pioneers is placed at the disposal of the 1st Battalion 53d Regiment, with which it will bivouac.

8. The machine guns will remain in their present position.

9. Supplies are being assembled at Matsui whence they will be sent up to the troops. Separate orders are being issued on this subject.

10. My position is at the bivouac of the 1st Battalion 53d Regiment. Issue of orders will be at 9 p. m.

The first perceptible result of these orders was that between 3 and 3.30 p. m., two or three Red companies advanced in the valley and along its northern edge against the weak spot at the northeast wing of the Blue position, and proceeded to cutflank and enfilade a badly-sited fire trench. These companies had crossed 660 yards of extremely difficult ground in a remarkably short time; they were now taken in flank by the enemy's guns, and, at the same time, were engaged by his reserve which had hitherto been stationed in rear of his right flank; they were therefore compelled to retire, but their daring advance taken in conjunction with the entrenching work which was being carried out by Red in front of the position, had the effect of drawing Blue's attention more towards his center, in rear of which he now stationed his reserve.

In the later part of the afternoon the Blue commander came to the conclusion that no fresh developments were to be expected on the 13th February, and at 5 p. m. he issued orders for the troops to bivouac on the position.

Meanwhile the attacking side had been carefully entrenching a position in front (northwest) of the bend of the railway; the intention was, apparently, to deceive the enemy as to the real objective of the attack; to occupy the position with the minimum number of troops, allowing the remainder to rest; and at daybreak to advance across some very difficult ground along the railway and thence against Nagao, in order to envelop the extreme right wing of the Blue Detachment.

The activity displayed by Red in front of the position naturally had the effect of making some action in this quarter appear probable. Blue fell into the trap, but at the same time advanced a portion of his infantry line during the night, so that, in the mornin, Red found the enemy in a different position from the one that had been reconnoitered the previous evening.*

The Director was justifiably proud as he showed me over the Red entrenched position. The work had been executed with extraordinary thoroughness in a remarkably short time; the trenches were well sited and every contingency had been provided for. The number of troops required to occupy the position had been reduced to a minimum; the remainder were accommodated in bivouacs nestling into folds of the ground in rear of the position. There

*See Director's criticisms on pages 666-667.—Ed.

was a well-equipped observing station with telephone wires leading in all directions, where orders were being quietly given out for the next day's operations.

The orders in question gave a description, as full as was possible, of the enemy's position, and dealt with the question of establishing communication with the left wing of the Red Army; they then continued as follows:

"The Detachment will to-morrow deliver an enveloping attack against the enemy's right flank.

"The 2d Battalion 53d Regiment, with the machine guns, will advance at 5.30 a. m. to-morrow across the valley, and will draw fire from the position, keeping the latter under its own fire, and endeavoring to attract as large a number as possible of the enemy's troops to the front of the position; a detachment will attempt to capture the enemy's battery.

"The 1st Battalion 53d Regiment, with one section of the Pioneers, will move at 5 a. m. by the valley east of Nagao, so as to outflank the enemy's right and attack him in rear.

"Two companies of the 2d Battalion 53d Regiment will follow in rear of the 1st Battalion and will be at my disposal.

"The cavalry will advance beyond Nagao and reconnoiter in front of our left flank.

"The artillery will open fire at daybreak (as soon as the infantry attack begins) against the enemy's right wing and against his guns."*

From 4 to 4.30 a. m. on the 14th February patrols could be seen working actively before the front and left wing of the Blue position. Nothing else was visible and the enveloping attack, which moved skilfully and silently across country, was unperceived; the proper working of the patrols in accordance with active service conditions was ensured by the presence amongst them of a number of young umpires.

Blue was fully aware of the weakness of his right wing, and had spared no pains to strengthen it; nevertheless at 5.30 a. m. on the 14th February the Red attack, advancing along the valley of Nagao (see Fig. 2, a-a-a) effected a complete surprise, and brushed aside all attempts at resistance. (The eastern sky was light at this time, but the lower ground was still wrapped in darkness.) Blue had altered his line of defence in the night and had pushed out two companies of infantry and some pioneers some 200 yards to a small knoll which was favorably situated for defence.

This unexpected alteration in the position caused Red some delay; the short burst of firing gave a general alarm, and the cohesion of the attack was destroyed. The Red attacking force encountered the Blue reserve, which had been hurriedly brought up, near the four small ponds on either side of the road 500 yards

*See Director's criticism quoted on pages 666-667.—Ed.

northeast of Nagao; hand-to-hand fighting would now have taken place right in the Blue artillery position had not the Directing Staff put an end to what had become an impossible situation.

Information was now conveyed by the Directing Staff to the Red commander that his main army was retiring, and that its left wing would be occupied till the morning of the 15th in crossing to the eastern bank of the Kizu River near Kamitsungo. The Kizu Detachment was to withdraw forthwith to the ground south of Terada (on the eastern bank of the Kizu River) covering the retirement of the left wing of the army and securing its left flank. (See general map, Fig. 1.)

Reds were thus unable to profit by the advantage which they had just gained, though their success assisted them in the difficult task of breaking off the engagement. They retired, showing their teeth more than once, across their position of the day before, through Tanabe, to the fords of the Kizu, south of Tono and Mizushi.

Blue now gained a little breathing space, and was in a position to comply with an order received from his main army, which read as follows:

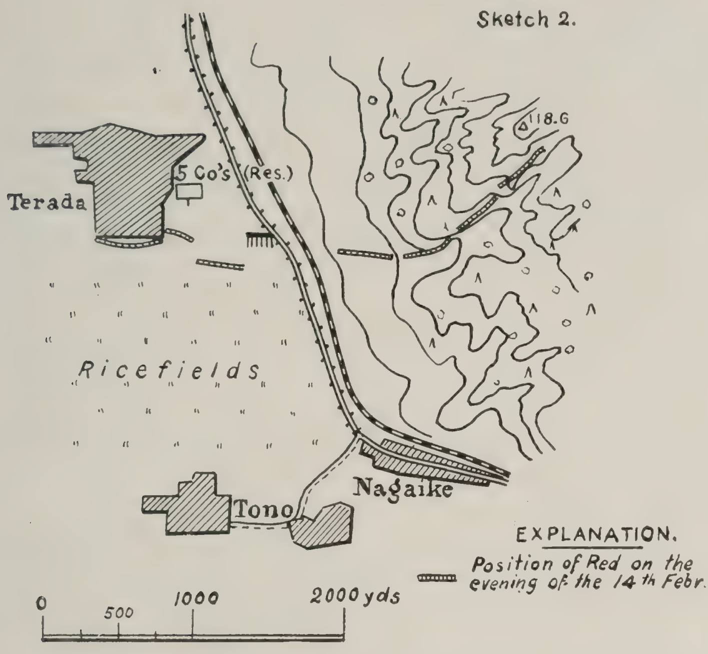
"The Blue Army will reply to the Red attack by a counter-attack on the morning of the 14th; with this object all available forces will be set in motion simultaneously. The intention is for the army to cross to the right bank of the Kizu at and near Kamitsungo about the afternoon of the 15th.

"The Kizu Detachment will forthwith drive back the enemy opposed to it, and will cross the Kizu River early to-morrow near Tono. It will then advance through Terada with the object of facilitating the passage of the river by the main army."

In accordance with these instructions Blue immediately gave orders for a pursuit. Three columns were formed, but they took a long time to get off and failed to cooperate. The cavalry and one company of infantry moved out some way to the south across the Fujisaka-Tanabe road. Red's retirement was, therefore, not closely pressed, and at Matsui, Asaoka, and Tanabe good opportunities presented themselves for counterstrokes against the pursuing force; by these means Red gained time to draw off towards Tono and to cross the river.

The River Kizu was forded by the main body and the artillery south of Tono, and by the rearguard between Asasano and Mizushi. This was the first occasion on which troops had forded the river, and the achievement was only rendered possible by the unusually low level of the water. The actual depth in the rapid part of the stream was not more than 25 to 27 inches at the ford, but the bottom was partly mud and sand and partly loose boulders. A careless step plunged a man up to the chest in the ice-cold water. The waves surged over the guns more than once. The pressure of the current was considerable, and the main branch of the river was quite 110 yards wide.

The fording of this river in the middle of winter, as an incident


Sketch 2.
Terada
5 Co's (Res.)
118.6
Ricefields
Tono
Nagaike
EXPLANATION.
Position of Red on the evening of the 14th Febr.
0
500
1000
2000 yds

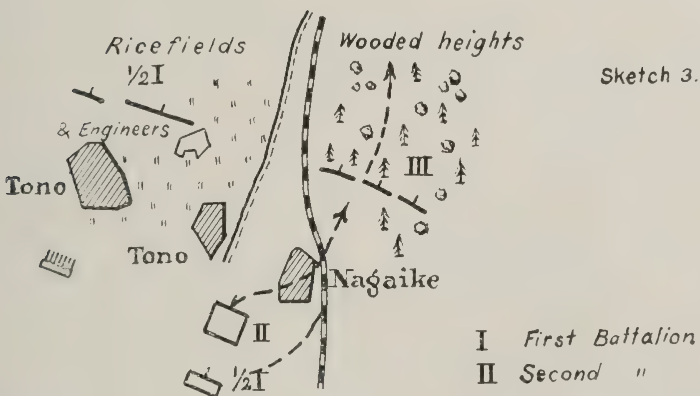

Ricefields
½I
& Engineers
Wooded heights
Sketch 3.
III
Tono
Tono
Nagaike
II
½I
I First Battalion
II Second "

Fig. 3.

in a tactical exercise which had involved at least a third of the troops bivouacking in the open, must be regarded as a notable achievement. The chief difficulty for the vehicles, in addition to the steepness of the banks, was a deep channel which runs along the eastern shore; but all obstacles were easily overcome.

The dykes on the east bank of the Kizu afforded Red a strong position; just south of Tono these dykes receded, thus enabling a battery to be brought into action. In this position Red was finally able to check any further pursuit. As, however, the Red outpost line was now strongly established on the line of the river, it was no longer possible for the Blue cavalry and infantry patrols to cross the stream by daylight.

Leaving the battery in its position, for the time being, Red now entrenched himself in a well-chosen position near Terada. (See sketch 2, Fig. 3.) The position extended from that place, which formed a *point d'appui* for the western flank, into the hills; the front was about 2,650 yards and was occupied by seven companies, eight machine guns and six field guns; five companies and the pioneers were held in reserve. The 1st (skeleton) Battalion of the 1st Regiment, which had arrived at Hirakawa, was placed at the disposal of the Red commander.

The Blue Force, which had reached the neighborhood of Tanabe, was likewise reinforced by the 1st Battalion of the 22d Regiment, which had taken up quarters for the night at Asasano. At 4 p. m. the Blue commander gave orders for the troops to occupy close billets in Tanabe. Outposts were pushed out as far as the left bank of the river, and touch was gained with the reinforcements at Asasano.

The Blue patrols had ascertained that Terada was still occupied by the enemy, and that hostile detachments were at Tono and Biwansho. The extreme flanks of the Red outpost line were located at Asajuroku and Mizushi. The Blue commander decided to cross to the eastern bank and attack the enemy early on the 15th; orders were issued for the assembly of the Blue troops at 4.30 a. m. at Tanabe and Takigi.

The Red Force finished entrenching by 6.30 p. m., and the troops went into close billets at Terada. An outpost company was pushed forward to Tono and another to Nagaike to watch the line of the Kizu. The Red patrols had found it impossible to cross the river in the afternoon, but it was established with the aid of telescopes from the hills near Terada that the Blue troops had entered Tanabe, but had not emerged from that place.

The Red commander, therefore, issued the following orders for the next day's operations:

RED.

Headquarters, Terada, 14th February, 9 p. m.

1. No further news of enemy. The 1st Battalion 1st Regiment has been placed under my orders.

2. The Detachment will to-morrow hold its present position in order to cover the retirement of the main army.

3. The cavalry squadron will move off at 4 a. m. to-morrow and, in cooperation with the outposts, will secure the passages of the river between Mizushi and Asajuroku, and establish communication with the main army, via Tsukahara.

4. The outpost companies will fire vigorously on any Blue troops attempting to cross the river. If compelled to retire by superior forces, the right outpost company will fall back first of all to our right flank, and then to the reserve; the left outpost company will fall back to the left flank of the position.

5. At 5 a. m. to-morrow the outposts will re-occupy their day positions. The O. C. 1st Battalion 53d Regiment will resume command over his outpost company as soon as the latter falls back from the line of the river.

6. The 1st Battalion 1st Regiment will be assembled at 5 a. m. near the reserve.

7. The baggage will be formed up at 6 a. m. in column of route, head at South-Hirono.

8. I shall be with the reserve after 5 a. m.

At daybreak on the 15th February Blue endeavored to ford the river in two groups, south of Tono and near Asajuroku. It was, however, 6 a. m. before the first troops crossed; endeavors made at 4 a. m. met with no success, as the conditions had altered and the river was too vigilantly guarded.

The Blue battery, which had come up to the river dyke southeast of Biwansho, was caught in a helpless situation by the fire of a company of Red infantry on the northeast bank, and was put out of action before our eyes. It was not possible for the battery to open fire rapidly on the narrow summit of the dyke with the teams in the intervals between the guns; the infantry escort remained under cover instead of replying to the enemy's fire. The group which was intended to carry out the secondary attack failed in its task, and exposed itself to unnecessary losses.

Meanwhile, however, the larger Blue group had forced a passage opposite Nashima in dashing style, wading through comparatively deep water and driving off the enemy's outposts at that point. This success compelled the remainder of the Red outposts, who had hitherto held their ground, to fall back from the line of the river.

By 7 a. m. Blue had obtained possession of Nagaike and Tono as *points d'appui* for a further advance. The battery was brought into action on the river dyke, near the road, to the south of Tono; the northern edge of Tono and a temple northeast of that place were occupied by two companies of infantry and by the pioneers, who had crossed with the guns.

The intention of the Blue commander was to attack through the wood with his right wing; with this object he organized his troops as shown in sketch 3, Fig. 3.

Too much weight was, however, attributed by Blue to the difficulties of crossing the somewhat swampy rice fields between Tono and Terada. As a result the frontal attack was not sufficiently pressed home, while a wide outflanking maneuver through woods and mountains to the eastward was undertaken by a force of ten companies. This maneuver did not long remain unperceived by Red, who left two companies and two machine guns in the eastern portion of his entrenched position and employed the whole of the remainder (ten companies of infantry and one of pioneers) in a vigorous offensive west of the railway, the machine guns advancing along the railway embankment itself.

Two or three Blue companies attempted a counterattack out of the wood, but, taken as a whole, the Blue force had obviously got out of control in the wood, while, on the other hand, the Red attack was seen to be so well combined, and to be making such good progress across the rice fields and railway track that its success appeared beyond doubt both to umpires and spectators.

I myself accompanied this attack on foot through the rice fields; at one moment we marched comfortably along dykes or over frozen places, at the next moment we sank waist deep; the dash and endurance displayed by the troops in this advance was beyond all praise.*

At the close of the maneuvers the troops were still fresh, though they had not had their clothes off for four days. Not a single man had fallen out from the 19th Infantry Regiment and only a very small number from the other units.

Operations had ceased at 9 a. m. The conference, which was long and exhaustive, was held on the dyke of Terada, in the Red position, at a point where a good view was obtainable; it was concluded by 11 a. m.

The 19th Infantry Regiment returned by rail to Tsuruga, arriving at 11.30 p. m.; the remaining units returned to their stations by march route. The troops from Fushimi had to march 10 miles; the 9th Regiment from Otsu, 14 miles; the 53d Regiment from Nara, 12 miles.

THE DIRECTOR'S CRITICISMS.

A few of the most important points in the Director's criticisms are given below:

As regards the first day's operations the Director criticised (a) the employment of the mounted troops as "Independent Cavalry," a rôle for which they were numerically far too weak; (b) the dispersion of the troops, on both sides, during the advance, and (c) the notion apparently entertained by the artillery that they should only come into action when definite targets were visible.

As regards the events of the 13th February, he spoke somewhat as follows:

"The retirement of Blue on this day was well executed, but de-

*For Director's criticisms on this day's operations, see pages 666-667.—Ed.

tachments should have been left in front of the position, for security; some such detachments were left, it is true, but they withdrew quickly on to the position itself.

"If your intention at the moment is not to strike a decisive blow, but to fight for time in which to strengthen your position, you must leave a screen of small detachments out in front, towards the enemy; these detachments must establish themselves at suitable points and keep off, or at any rate impede the enemy's reconnoitering parties. There is, of course, one drawback to the use of these detachments, they may become advanced positions; but in mountain country like this that is a matter of no importance.

"The Red side did not pursue with sufficient energy; they should have used their whole force, instead of employing isolated detachments. If Red had pressed the retirement vigorously Blue might have had great difficulty in reaching his position in the hills.

"In defensive positions in mountain country the question of fire at close ranges must be given the first place; fire at long ranges is of far less importance because the enemy will nearly always be able to get out of it in a short time."

The preparations made, by Red, during the night, for the attack on the morning of the 14th were completely approved by the Director, who rightly attributed the successful execution of these movements—in spite of all the difficulties of the terrain—to the thoroughness of the training which the troops had received.

On the other hand, some fault was found with the handling, on the same day, of the Blue reserve, on the ground that it was not used *as a whole* for a counterattack, but was employed to reinforce the front at various points.

The further steps taken by both sides on the 14th (that is, the retirement of Red across the Kizu, and the pursuit by Blue as far as that river) were approved by the Director. His remarks on the last day's operations are sufficiently interesting to justify me in giving the principal points verbatim.

"On the afternoon of the 14th the Red commander found himself compelled by the general situation and by the nature of the ground near Terada to act on the defensive. But, although the position was strong for defence and the tactical situation might have been thought to prescribe a purely passive attitude, the possibility of an active defence was by no means excluded. Our New Japanese Infantry Training inculcates the importance of being always on the watch for opportunities of taking the offensive, or of delivering a counterattack, even during the most passive phases of a defensive action.

"In this case such an opportunity was afforded by the woods east of the road; it was therefore a better plan to fix bayonets for wood fighting than to construct any number of trenches and wire entanglements. In wood fighting, as in night operations, events move rapidly; after a short or vigorous fire-fight the troops must

charge with cold steel; similarly provision must be made for launching a counterattack.

"The Blue attack on the last day was a failure. If you throw troops into a wood they are soon swallowed up in it and control is lost; the Blue commander only kept two companies at his own disposal, the whole of the remainder had been thrown into the wood.

"If the intention was to attack through the wood the enemy should have been engaged by at least one battalion on the front. Nagaike-Tono. I imagine that the reason for the plan of attack adopted was the belief that the rice fields were impassable; but infantry must be able to get over any kind of ground; certainly no ground which is capable of being traversed at all can be considered as affording protection merely by the difficulties which it presents."

In conclusion, the Director was able to testify with justifiable pride "That both officers and men had maintained their zeal, undaunted by the difficulties they had encountered, and had spared no exertions to render the maneuvers a success."

THE AUTHOR'S OBSERVATIONS ON JAPANESE TACTICS.

The object, which was systematically pursued in these winter maneuvers, had been to employ the troops under the most varied conditions and at all hours of the day and night.

The Japanese views on night operations, which are based on their experiences in the war, are, in the main, approved in our (German) army, and form the basis of the British instructions for night operations, and also of French military instructions on the subject. The Japanese instructions lay repeated stress on the importance of adapting all measures to the circumstances of the case; the dispositions should be such as to compel the enemy to cross the skyline "and show himself upright;" on the other hand, their own troops must stand up as seldom as possible.

Troops should keep a little way off roads unless the latter are required for purposes of movement; light backgrounds are to be avoided.

By systematic training of the men's hearing powers, frequent night operations, and inspection of individual units by night, the troops acquire a capacity for scouting in a noiseless, cat-like manner, which makes the work of Japanese infantry patrols as interesting to watch as the movements of a fox approaching its prey. Instinct and technical training work hand in hand. On one occasion an attack, which was delivered at daybreak over difficult ground by six companies, advanced so swiftly and silently that some reserves, which were stationed only 200 yards away, were unable to come up in time to assist in the defence.

The most important lesson which I mastered during these winter exercises—a lesson of which I had learned something at the autumn maneuvers, and even before them—was in connection with that most important question of tactics—extensions under fire.

The Japanese demonstrated with the blood of many of their race

that their divisions were capable of occupying areas and extents of front in battle to which their European opponents would not venture to assign formations of less than an army corps; they have, therefore, come to assume both in the education of their officers and in the training of their troops that they can safely exceed, by a considerable margin, the extensions which are regarded as normal by their European instructors. *We* (Germans) may think they are wrong, but we must nevertheless assume that we shall be opposed on a front of 1,550 yards of entrenched position by only four companies, four machine guns, and six guns employed in dispersed sections; or, on a front of 3,850 to 4,400 yards on the flank of an army, by only one division, which would nevertheless offer a very serious resistance. We ought, therefore, especially in view of the great expenditure of ammunition entailed by the increased range of modern firearms, to consider the matter impartially, and revise our preconceived opinions; or else we shall have to admit that a foreign army is superior in *morale* and fighting value to our own. There is no middle course if we propose to continue our policy of giving the doctrine of the enveloping attack the foremost place in the training of our officers and men, and if we do not decide to adopt the doctrine of "penetration" as better suited to our dense maneuver and battle formations. The envelopment of these thin widely extended fronts, occupied by troops of a fighting value equal to our own, would, as a rule, be impossible in practice.

The Japanese troops derive great help in carrying out this doctrine of wide extensions, not only from their *morale* and spirit of self-sacrifice, but also from their skill in making use of the ground and overcoming natural obstacles, and from their excellent training in close reconnaissance, as well as from their rapidity of movement, which greatly increases the value of the reserves. To these must be added their perfect mastery of the use of entrenching tools both in attack and defence* and the marvelous skill displayed, even by non-commissioned officers and men acting on their own initiative, in the application of field fortification.

There is also something of an artist's touch in the eye for ground, which enables the leaders to get the best value out of the use of field entrenchments, and in their capacity for passing with lightning rapidity—undeterred by any consideration of the labor which has been expended—from an apparently passive and apathetic defensive to an offensive of the most vigorous and active character. In this they are greatly assisted by the capacity of their infantry battalions, regiments, and brigades for covering distances of as much as 4,400 yards at a run.

No labor expended on field works is considered as excessive, even if they are only likely to be required for a short time, or even

*In this connection it must not be forgotten that the Japanese have a liberal allowance of pioneers; each division has a pioneer battalion of three companies.

when the men working on them are aware that they are merely intended to mislead the enemy.

In spite of some mistakes, such as must always occur, it could be seen that the science of field engineering was thoroughly mastered, understood and intelligently applied by all ranks down to the individual soldier. Indeed, it was observable that as regards skill, theoretical knowledge, and general method of setting to work, the individual Japanese soldier was better trained in field fortification than our own; the men were allowed considerable independence in the execution of details and in improvising makeshifts.

The following are a few of the characteristic points which came to my notice during the maneuvers:

(a) Defensive works visible from a great distance are, as a rule, assumed by the attackers to be dummy trenches, and no fire is directed against them, in the first instance, until it becomes evident, during the attack, that they are occupied. (All genuine works are tested beforehand, whenever possible, by the side which constructs them, to see if they blend well with their backgrounds, so that the real trenches are seldom visible from a distance.)

(b) It is considered very unlikely that an attack at dawn will find the defenders in the same position as they were in when reconnoitered on the previous evening. By day, works with a wide field of fire are considered the best; by night, preference is given to those which compel the enemy to cross a skyline or to show himself against a light background.

(c) No one shrinks from the labor of constructing two or three lines of defence, so as to allow for alternative positions in special circumstances (*e. g.*, darkness, thick weather, etc.).

(d) Two or three hours, even in complete darkness, is considered long enough to make ground, which affords ordinary facilities to the advance of an attack, practically impassable to the enemy; and this must be effected without letting the enemy know that there is any reason to modify the favorable reports received from his reconnoitering parties on the previous evening.

(e) As regards entrenchments, whether in attack or defence, *one line* of resistance is preferred; this line must, however, have salients and reentrants, and contain gaps which are swept by fire from other points; it must present to the enemy's reconnoitering parties an inaccessible zone of greater or less width, in which the defenders' fire prohibits any closer view.

Bodies of troops on the march are, as a rule, masked, at a distance of 550 yards, on either flank, by a screen of infantry patrols; these patrols have to cover a great deal of ground.

(f) When the enemy has made all preparations for a formal attack, and can be seen at nightfall entrenching and digging himself in, one should suspect a surprise attack will be delivered from a quite unexpected direction, most probably in some quarter—if such exists—where the difficulties of the terrain offer a deceptive guarantee of protection.

I have seen the Japanese infantry at work for a long time, but I have never yet seen the ground which they would fail to cross in the long run. The only reluctance they display—as a burnt child dreads the fire—is towards crossing open ground bare of cover. In such cases they prefer to go greater distances through marshes or forests.

(g) It is a mistake to suppose that entrenching imposes more labor on the troops in the long run, than the attempt to dispense with it.

When the sentries, pickets, and detached posts have been made secure behind entrenchments and obstacles, it is possible to practice the most extreme economy of force among the troops in rear. The latter, as well as the staffs, in their well-concealed bivouacs, eat bathe, rest, or carry on their work in complete tranquillity; communication by telephone, flag, or lamp is established with neighboring bodies of troops and with headquarters, and supplies of the simplest, though most serviceable kind are issued in ample quantities. Troops were limited to the active service scale of luxuries, but these were made full use of; practically no restrictions were placed on the utilization of the resources of the theater of war.

Though these maneuvers took place in winter the greater part of the troops bivouacked on three successive days; I think I now understand the attitude of the Japanese towards bivouacking. They regard it, in all its various graduations from bivouacking proper to close billeting, as a natural incident of field warfare at any time of the year. They do not like it—quite the reverse—but they treat it as an unavoidable necessity; they make themselves quite at home in their bivouacs; they wash, unpack their kits, complete deficiencies from the baggage, which comes up just as it would in war, rest themselves thoroughly, and renew their strength.

In our (German) army—to be quite candid—bivouacking is regarded rather as a pause between two stages of the maneuvers, and not as an opportunity for a thorough rest; this is not looked for till the next rest-day in billets, upon which we are, in consequence, unduly dependent.

The most brilliant achievement during these exercises was the fording of the Kizu River; this river seemed to me to resemble the Lech in strength and velocity of current while the volume of water on the day in question was like that of the Alz; the water was covered with ice, except in places where the strength of the current had prevented it from freezing.

The stream was piled up to such an extent by the passage of the artillery that the waves broke over the guns; the gun leaders were harnessed in a simple and practical manner and the driving was good and careful all the way across. At some places the infantry had to wade breast deep through the foaming current; at these places the men linked arms chiefly as a precaution against the giddiness to which some individuals are liable at the sight of swiftly flowing water.

The dismounted company commanders of the infantry had to

wade, as did the medical officers, pay officers, etc. Some mounted officers also waded through out of zeal. Fighting went on after the crossing was over, until the tactical situation allowed an opportunity for the men to dry themselves and change their underclothing, after which they went on outpost duty or into bivouac.

The employment of dismounted company commanders—a plan adopted by Napoleon—appeared to me, in the light of these maneuvers, as a severe but eminently sound arrangement; it serves to bring out the hardiest and fittest infantry officers, and in cases (such as in pioneer battalions in this country) where there are six or eight horses available on the battalion staff to mount the officers if necessary, the system appears to be open to no objections.

Very noticeable at these maneuvers was the active support given by the administrative authorities, as was also the interest displayed by all the schools. As early as the first issue of orders at Tamamizu we met the Head of the Administration (*Regierungs präsident*) of Kyoto, a very distinguished old gentleman speaking French and German fluently, who did not think it at all beneath his dignity to go in advance with his councillors and officials and see that everything had been arranged for the troops. The Head of the District (*Kreischef*) and the official in charge of bridge construction (*Flussbauchef*) were present from 3.30 a. m., on a bitterly cold morning, on the day we crossed the Kizu.

The schools followed the troops in the same way as is done at the autumn maneuvers, that is to say, each class devoted a complete day to following some military movement which was going on within reach. Everything that was happening was carefully and patiently explained to them; they were shown the field entrenchments, the bivouacs, etc.; they in their turn displayed the liveliest interest, which will not fail to bear fruit later on. The pupils of the Secondary School (*Mittelschule*) at Tono-Nagaike were also given leave to follow the exercises, and later to prepare a good reception for the Directing and Divisional Staffs at their billets.

I feel that I can not conclude this report without some word of thanks to those through whose courtesy I was enabled to get so much pleasure and advantage from these maneuvers, and especially to the gallant commander of the division, whose kindness was unremitting, and whose uppermost thought was that the guest of his army should see everything that was possible as far as was compatible with the interests of his country. The Director also showed me the greatest kindness, in spite of the numerous demands on his attention, and his assistants were equally attentive. All orders and instructions were conveyed to me as early as possible, and any information which I asked for was promptly given. The civilians, both officials and people, also displayed a cordiality towards the foreign guest which far surpassed what is customary in our own country on similar occasions.

Taken as a whole, these winter exercises, in spite of their modest scope, were the most instructive of any that I witnessed during my attachment in Japan.

Cross Sections of Breakwaters to Withstand Wave Action

BY

Col. FREDERIC V. ABBOT
Corps of Engineers; Member American Society Civil Engineers

Two prime factors, exposure and cost, determine the best form of cross section to adopt for a breakwater. Exposure varies with the fetch of waves and with the depth of water at the structure and the front between it and the open ocean. At Charleston, S. C., with moderate depths and outlying shoal areas to break the full force of the ocean waves, a rubble mound cross section has served well for the jetties, built to deepen the channel, but serving also as breakwaters to reduce the force of waves in the jetty channel. In construction care was taken to avoid using small stone for a hearting and covering it with selected large stone, as has been frequently done in Europe. Run of the quarry was used up to about low water, the drilling and blasting being conducted so as to produce as large a percentage of stone of 7-ton weight as possible. Above low water level a skeleton was made of stone from 1 to 7 tons in weight dropped directly along the axis up to a height somewhat above high water level, and left with side slopes as steep as they would stand; this skeleton was covered with quarry run also dumped directly along the axis with natural side slopes up to a height of 10 or 12 feet above high water. The first heavy storm would knock this grouting and the upper part of the skeleton down, beating the smaller pieces in among the undisturbed larger pieces of stone forming the skeleton, and distributing the balance on the slopes. The grouting was then replaced to the former height, taking this time much less stone than at first, and a new storm was awaited. It generally proved to be necessary to repeat this process three times before the cross section assumed a final permanent form, with crest approximately at high water level. Writing from memory only, it is the writer's impression that at the outer ends of

Fig. 1. Drilling holes for 30-ton capstones, Sandy Bay breakwater. Three photographs were fitted together to make this illustration complete.

the Charleston jetties, 20,000 feet from shore, the mound was ultimately nearly 90 feet wide at mean low water with a crest about 5 feet above; the crest, which was very smooth and level, was about two-thirds of the way from the seaward to the harbor side of the 90 feet lying between low water levels on the two sides. At depths greater than 15 to 25 feet on seaward side, depending on the local protection afforded by outlying shoals, the slopes were nearly 1 on 1, and from those depths to the crest the seaward surface was convex toward the sea, the slope becoming very gentle for the 2 feet immediately below the crest. The upper surface was singular-

Fig. 2. Sea face of western arm, Sandy Bay breakwater, looking east-southeast.

ly uniform and smooth, considering the great size of many of the stones. The harbor side was similar but much steeper throughout. This form of cross section permits waves to mount the slope and fall over into the harbor with the least abrasion and injury to the structure, and it has resulted from the action of the very forces it is intended to resist. The engineer has simply supplied material, which waves have molded into final shape. The stilling effect of such a structure is not as great as that of one designed to throw the entire wave back to the front, but, except in very heavy weather,

the water between the Charleston jetties is not rough enough to inconvenience any vessel adapted to ocean traffic. An advantage of this method of construction is that each part of the structure has a cross section just sufficient for its individual exposure, and therefore no stone is wasted to give a section stronger than is needed; repairs are extremely easy if a shifting of the outer shoals exposes any part of the superstructure to heavier seas than existed there when it was built. The last stone was deposited on the jetties about twenty years ago, and I am told that there has been no sensible degradation of the crest level in that time, although a

Fig. 3. Harbor side of Sandy Bay breakwater, looking south-east

number of West Indian hurricanes have passed over the harbor and done vast damage to the city and to shipping in the inner bay.

The Galveston jetties were originally built with rather small stone hearting covered with heavy stone sides and cap; the storm of 1901 produced disastrous effects; as soon as the capping or side covering was broken the interior washed out and the whole structure ravelled and collapsed for considerable distances.

At the Columbia River jetties the exposure and outlying depths are so great that a random cross section somewhat similar to that at

Charleston has so far proved impossible to maintain above mid-tide without using enormous quantities of stone. Failures prove more than successes, and it is possible that at that point some form of cross section like the Colombo sloping block or the Sandy Bay coursed squared stone might have been cheaper in ultimate cost. The preparation of a suitable flat base for such an accurately laid superstructure would, however, have been vastly difficult and costly on the Columbia River bar.

At Point Judith, where the exposure is perhaps comparable with that at Charleston, a moderate rubble mound, with a superstructure

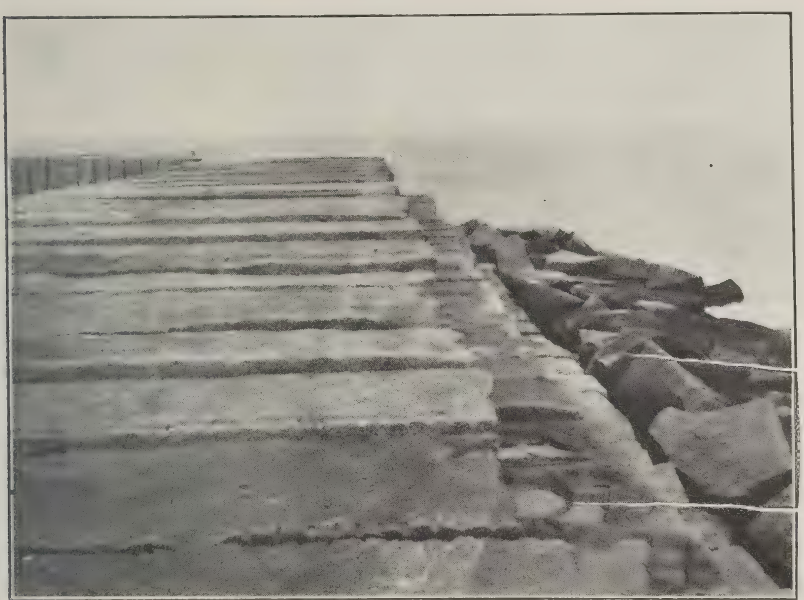


Fig. 4. Western arm, looking north.

of large blocks set to form a regular cross section, with predetermined surfaces of heavy rubble on the top and seaward side, has stood for years with comparatively small repairs, but when they have been necessary the regularity of the structure has made them costly because a number of stones, only slightly displaced, had to be taken up and reset to restore the bond and to avoid conspicuous irregularities. Where lines are straight, or surfaces are planes, the eye is quick to detect any elevation, depression, or bulge, and this is a serious disadvantage of too regular outlines in breakwaters where the forces in action are so great as to make deformations probable. Where integrity of the structure depends

on bond or a smooth surface for waves to mount on, such deformations are not only very unsightly but seriously impair capacity to withstand subsequent storms.

At Gloucester, Mass., a very carefully built breakwater, with a superstructure of extraordinarily narrow cross section (side slopes of 1 on 0.7), has safely withstood storms of great severity and waves of great force, but the breakwater does not receive the unbroken force of the heaviest waves. It has recently received a wide sea apron of 5-ton rubble, to break up the force of waves before they strike the superstructure proper. In January, 1913, a storm moved

Fig. 5. Sea face of western arm, looking north-west.

a number of cap stones. They have been reset, and dowelled to the course below, to prevent sliding.

At Sandy Bay conditions are extreme. For long distances the depth of water is 80 feet; the exposure is to the northeast, from which direction come the heaviest storms; there are no outlying shoals to protect the part of the breakwater lying in the deepest water. A heavy rubble mound forms the base of the structure. An attempt to bring it up to 18 feet above mean low water (9.4 feet above high water) by simple heavy random blocks failed on the first attempt, and the experiment was not repeated. It is to be re-

gretted that it was not pushed further, for it is possible that ultimately, as at Charleston, S. C., a cross section would thus have been secured exactly adapted to the exposure. The depth is so great, however, that an enormous quantity of stone would have been needed to give sufficient base for the part affected by wave action, which here extends to 30 feet below mean low water, as shown by actual surveys. In designing a superstructure for this breakwater, all engineers have attempted to keep the width of foundation a minimum on account of the great quantity of stone needed to fill out the slopes of the substructure in 80 to 90 feet of water. From

Fig. 6. Harbor side of breakwater, looking north-west.

the bottom up to a depth of 30 feet below mean low water the mound stands stable with slopes of 1 on 1; from that level up to mean low water heavy rubble seems to stand safely at about 1 on 2. The first experiment with closely laid, uncut dimension stones was to set by divers on the seaward side a row of heavy rectangular headers at level —12 referred to mean low water, and a similar row at level —6 on the harbor side, filling between the rows with medium heavy rubble. Successive rows of dimension stone blocks were laid on these foundation rows, so as to give a stepped seaward slope of 1 on 2, and a stepped harbor slope of 1 on 1 up to level +17,

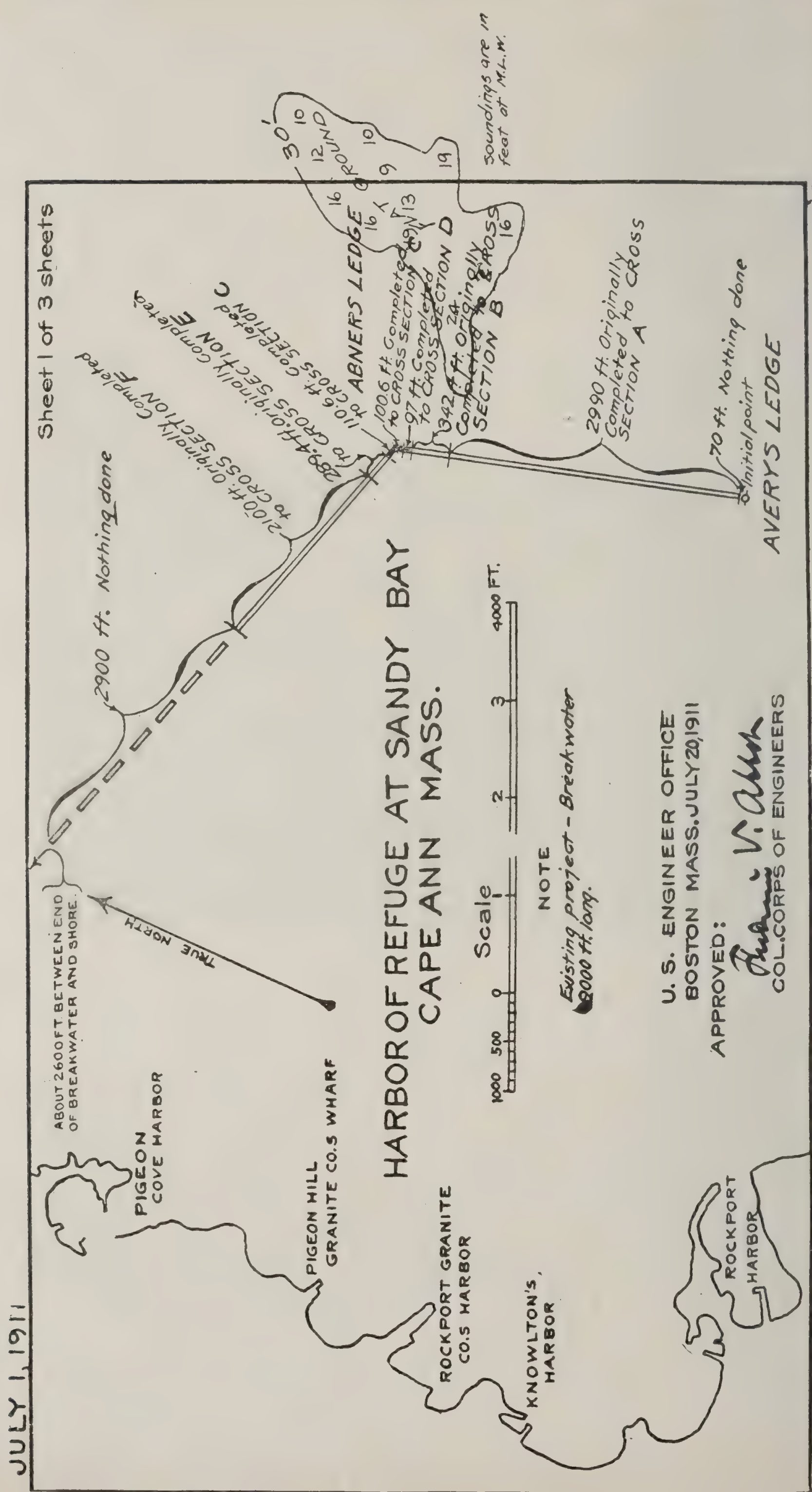

JULY 1, 1911
Sheet 1 of 3 sheets
HARBOR OF REFUGE AT SANDY BAY
CAPE ANN MASS.
Scale
1000 500 0 1 2 3 4000 FT.
NOTE
Existing project - Breakwater 9000 ft. long.
U. S. ENGINEER OFFICE
BOSTON MASS. JULY 20, 1911
APPROVED:
COL. CORPS OF ENGINEERS
2900 ft. Nothing done
2100 ft. Originally Completed to CROSS SECTION F
289.4 ft. Originally Completed to CROSS SECTION E
110.6 ft. Completed to CROSS SECTION C
100.6 ft. Completed to CROSS SECTION
97 ft. Completed to CROSS SECTION D
342 ft. Originally Completed to CROSS SECTION B
2990 ft. Originally Completed to CROSS SECTION A
70 ft. Nothing done
Initial point
ABNERS LEDGE
GROUND
AVERYS LEDGE
Soundings are in feat of M.L.W.
ABOUT 2600 FT. BETWEEN END OF BREAKWATER AND SHORE.
TRUE NORTH
PIGEON COVE HARBOR
PIGEON HILL GRANITE CO.S WHARF
ROCKPORT GRANITE CO.S HARBOR
KNOWLTON'S HARBOR
ROCKPORT HARBOR

Fig. 7.

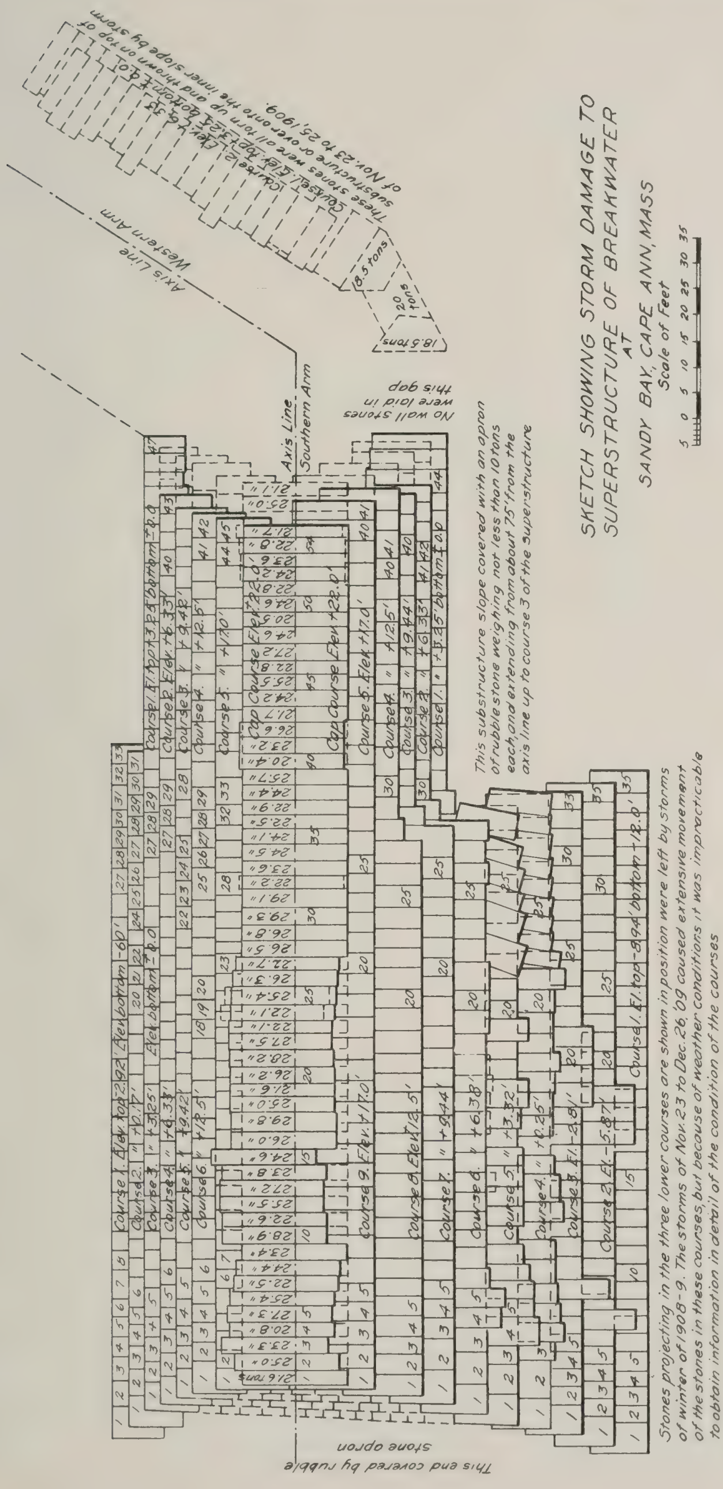
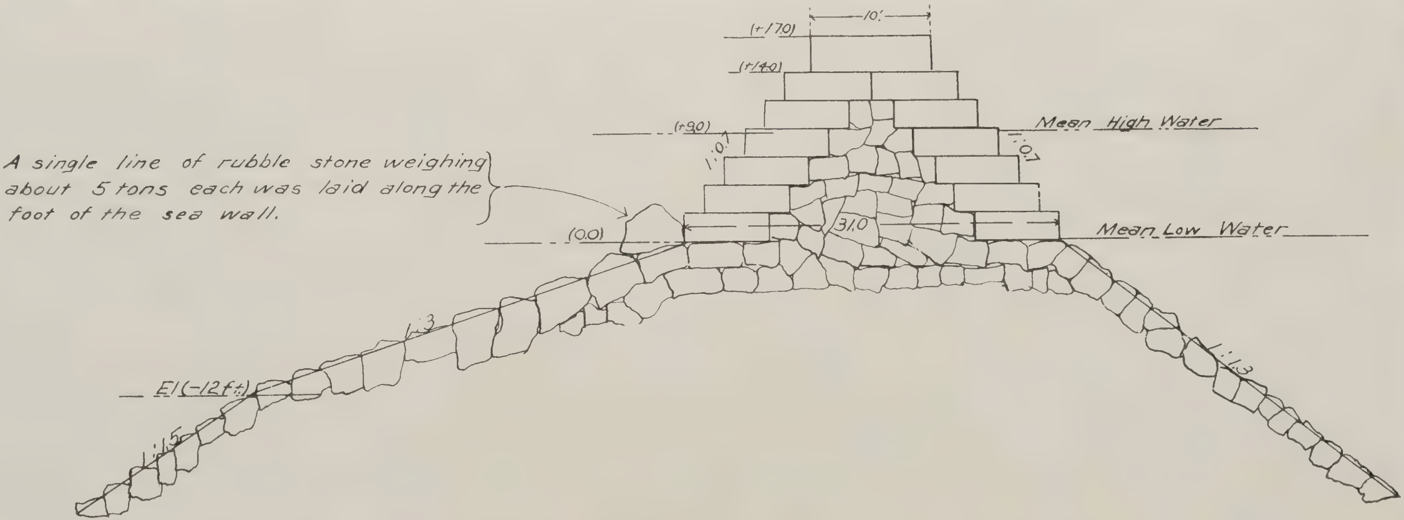

SKETCH SHOWING STORM DAMAGE TO
SUPERSTRUCTURE OF BREAKWATER
AT
SANDY BAY, CAPE ANN, MASS
Scale of Feet
5 0 5 10 15 20 25 30 35
These stones were all torn up and thrown on top of substructure or over onto the inner slope by storm of Nov. 23 to 25, 1909.
Axis Line
Western Arm
Southern Arm
No wall stones were laid in this gap
This substructure slope covered with an apron of rubble stone weighing not less than 10 tons each, and extending from about 75' from the axis line up to course 3 of the superstructure
Stones projecting in the three lower courses are shown in position were left by storms of winter of 1908-9. The storms of Nov. 23 to Dec. 26, '09 caused extensive movement of the stones in these courses, but because of weather conditions it was impracticable to obtain information in detail of the condition of the courses
This end covered by rubble stone apron

Fig. 8.

Fig. 9. *Cross section of Dog Bar Breakwater, Gloucester, Mass., as constructed.* Scale: 1 inch equal 16 feet.



NOTE: *Caps* are 3 foot rise, vary in width of face from 3.2 feet to 4.6 feet and weigh 8 tons to 11.5 tons with an average weight of 9.25 tons. *The walls* are in six courses in a rise of 14.0 feet or theoretically 2.33 foot rise per course. In construction the stone was quarried 2.2 foot rise for the first 5 courses, and the 6th course was cut to the rise necessary to secure the approximate grade of bed for the caps, resulting generally in slightly heavier stone in the 6th course.

The length of stone in the walls was specified to be not less than 7 feet nor more than 8 feet. The dimensions and weights of stones computed on an average length of 7.5 feet are as follows:

	Sea Wall.	Harbor Wall.
Maximum width of face	5.6 feet	6.0 feet
Minimum width of face	2.5 feet	2.2 feet
Average width of face	3.4 feet	3.2 feet
Weight of Maximum Stone	8.2 tons	8.7 tons
Weight of Minimum Stone	4.0 tons	3.2 tons
Weight of Average Stone	4.9 tons	4.7 tons

Substructure: Specifications required that in the seaward slope no stone should be used weighing less than 3 tons and that average weight of stone in each cargo should be not less than 4 tons. In rubble mound generally and harbor slope the stones ranged in weight from 500 pounds to 4 tons,

where the two rows were only about 5 feet apart at the inner ends of the headers. Capstones, 20 feet long and 5 feet rise, were set to bond the two walls. This superstructure withstood several winter storms, but in 1908 displacement of stones (see the illustrations herewith) showed that for a permanent superstructure the cross section was insufficient.

The latest form is due to Col. Edward Burr, who kept the harbor side of the superstructure unchanged, except that the lowest course is now at low water and not 6 feet below. The capstones are as before, but each is supported against sliding toward the harbor by

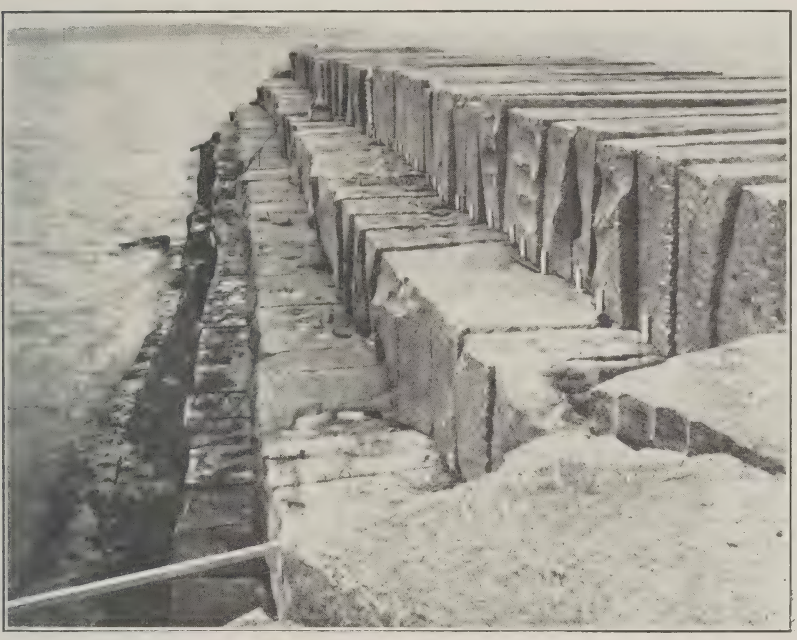


Fig. 10. Harbor side of western arm, showing dowel pins.

a 2-inch dowel pin set in a hole in the underlying course of the harbor wall, and solidly grouted in place. Below the cap the seaward face is built in steps sloping 1 on 1 and not 1 on 2, with the main object of giving greater lap of the upper stones on those in the lower courses, to resist the tendency to pull out to the front—the way in which the prior section failed most signally. In both harbor and land walls he stopped the dimension stone steps at mean low water in order to secure better inspection of the bedding of those most important blocks. The berm created by drawing back the foot of the seaward slope was utilized by using it as a shelf to

hold 10-ton rubble, which was brought up to high water level in front of the dimension stone. The rubble tends to shatter the waves before they reach the superstructure proper.

In 1909 a short section of this new superstructure withstood almost unchanged the heaviest storms of the past twenty years, and in accordance with recommendations of the Department, Congress has appropriated sufficient funds to build 800 feet of this cross section, of which 200 feet are on the western arm, and 600 feet are on the southern arm. This will be a sufficient length to develop the full destructive force of waves to which the southern arm is exposed, and if it prove sufficiently resisting the rest of that arm can safely be built to that cross section. There is an outlying shoal, submerged from 30 to 10 feet in local peaks, which gives that arm some protection. The western arm, however, crosses a wide, deep depression in which there are no depths less than 80 feet between the structure and the ocean to the northward and eastward; here the exposure will be the maximum, the fetch being the whole width of the Atlantic Ocean, and experience gained in the work will be the only safe guide for the officer charged with building the part of the superstructure opposite to this depression. The writer's own inclination would be to make the superstructure like the rest of the work, if found practicable, but to widen out the berm and the rubble front slope by the deposit on that side of large quantities of the heaviest granite rubble that could be bought.

The writer believes that Congress has been wise in suspending work on the superstructure of Sandy Bay until such time as an exceptionally violent easterly storm of some days' duration gives further data on the sufficiency of the present form, which has safely resisted all waves that have so far come against it.

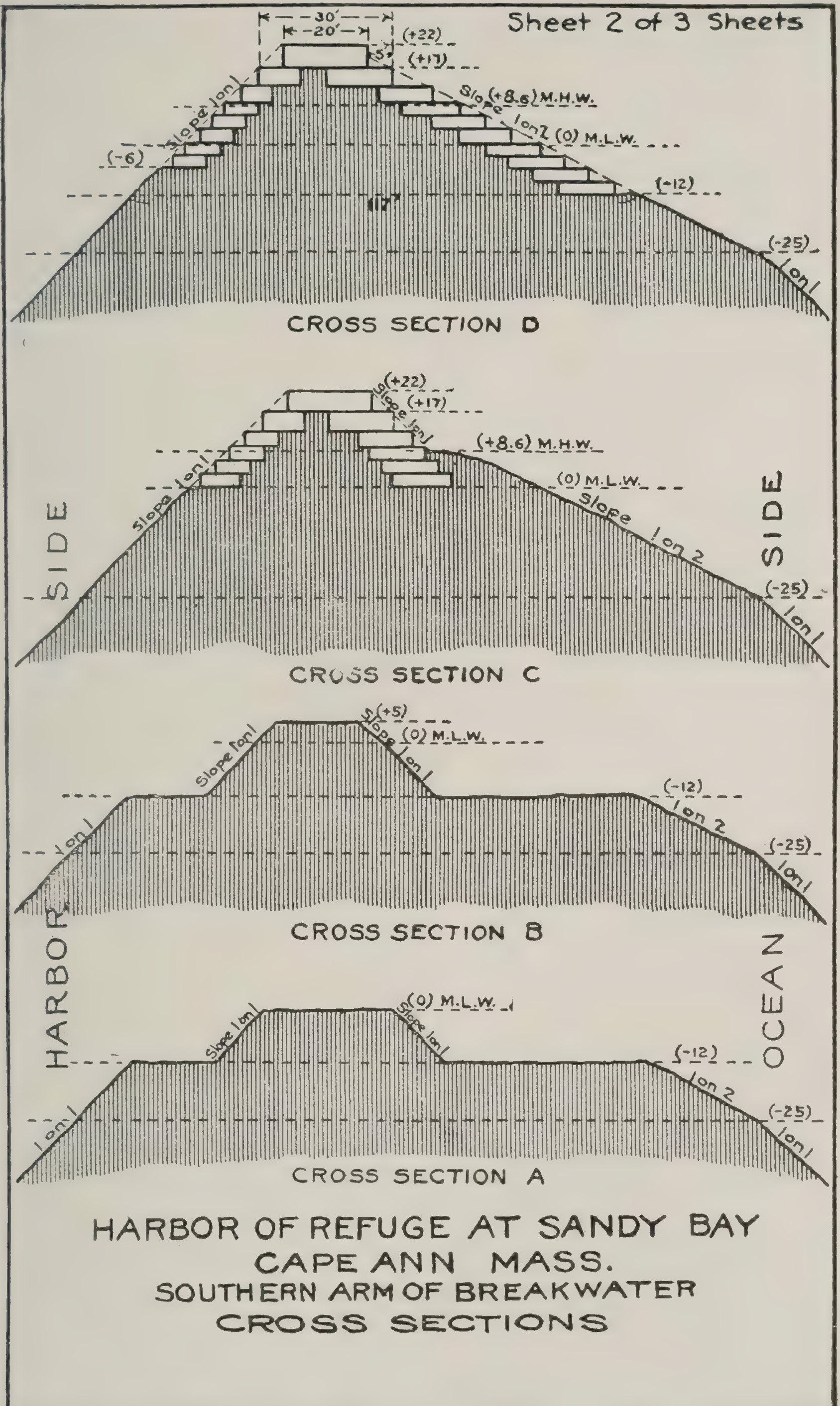

Sheet 2 of 3 Sheets
30'
20'
(+22)
5'
(+17)
Slope 1 on 1
Slope 1 on 2
(+8.6) M.H.W.
(0) M.L.W.
(-6)
(-12)
(-25)
1 on 1
CROSS SECTION D
(+22)
Slope 1 on 1
(+17)
(+8.6) M.H.W.
(0) M.L.W.
Slope 1 on 2
(-25)
1 on 1
CROSS SECTION C
(+5)
Slope 1 on 1
(0) M.L.W.
(-12)
1 on 2
(-25)
1 on 1
CROSS SECTION B
(0) M.L.W.
Slope 1 on 1
(-12)
1 on 2
(-25)
1 on 1
CROSS SECTION A
HARBOR SIDE
OCEAN SIDE
HARBOR OF REFUGE AT SANDY BAY
CAPE ANN MASS.
SOUTHERN ARM OF BREAKWATER
CROSS SECTIONS

Fig. 11.

Building a Ponton Bridge in Swift Water

BY

Capt. T. H. Dillon
Corps of Engineers

In the fall of 1912 the field instruction of Co. E, Second Battalion of Engineers, consisted in handling the ponton bridge material, including both reserve and light equipages and trestles. Before beginning this work a careful schedule had been laid out under the direction of the commanding officer of the Second Battalion of Engineers, which, as stated below, was carried out with certain minor exceptions.

The following description of the work is the report in full, made upon the completion of the work.

I. ORDER OF INSTRUCTION.

Laying out and explaining equipment, making lashings, loading and unloading wagons.

Transport one heavy division from depot to Merritt Lake, park and unload equipment.

Instruction in rowing wooden boats—still water.

Construction of bridges on Merritt Lake.

Transport light train to Merritt Lake.

Assembling and dismantling canvas boats, and construction of bridge.

Caulking wooden boats, as necessary.

Transportation of bridge material to Missouri River at suitable place near old Kickapoo Ferry landing.

Instruction in rowing and handling of single boats and rafts, construction of flying ferries, testing and placing of anchors.

Construct bridge across main channel.

Leave bridge in place and give instruction in duties of bridge guards and maintenance.

Dismantle bridge and get up anchors.

Transport all material except canvas boats by rafts down river to point near M. P. R. R. station.

Return material to depot.

Practice march of one week, including several crossings.

The above scheme of instruction as first laid out was practically adhered to, except that there was no work with canvas boats on Merritt Lake, and the time intended for a practice march was used in constructing a bridge across the river near the railroad station.

The dates, inclusive of this instruction: September 3 to November 15, 1912.

Average strength of company, 121; average attendance at drill, 54.

The teams and teamsters were under control of the battalion quartermaster.

II. RIVER CHARACTERISTICS.

a. Stage: 10 feet above low water at Kansas City.

b. Current: Maximum downstream current about 7 miles per hour; upstream current in eddies, about 2 miles per hour.

c. Depths: In eddies near concave banks, 20 to 24 feet; main channel, 6 to 12 feet.

d. Bottom: Soft mud in eddies; hard sand in main channel. Bottom shifts rapidly; in two days a shoal 200 yards long and 2 feet out of water was cut away and the depth of water increased to 6 feet. In three days position of main channel shifted 600 feet, and water 18 feet in depth shoaled to 6 feet.

e. Drift: Very little when river is going down, but a rise of 1 or 2 feet releases large quantities. Drift is very hard to dispose of on account of velocity of current.

III. HANDLING WOODEN BOATS.

Crew of six men and one steering oar required.

For work in swift current it is most important that bow of boat always be kept pointing almost directly upstream.

Principle of flying ferry used in handling individual boats.

In current up to 5 miles per hour, a good crew (7 oars) can row straight upstream or hold their own.

In 6-mile current a good crew (7 oars) can row straight across by using principle of flying ferry.

In swift currents, crews must use extreme care to keep well above

the bridge to avoid dropping down on the bridge and swinging across the current. Small skiffs must also be handled with care. A skiff held broadside on against a 4-mile current would be immediately swamped. A ponton boat supported broadside on from downstream side, as in laying across front of another boat, would be in great danger in a 6-mile current.

In an emergency, due to shifting of main current, it was necessary to place several additional anchors. Skiffs were sent out well upstream, and an anchor was cast as skiff floated down. By holding to anchor cable, skiff was let down to bridge with intention of allowing skiff to pass under the bridge between boats. This opera-

Fig. 1. Looking toward Kansas shore; note swift current.

tion was very dangerous, but was successful in most cases. In three cases the swinging of the boat from side to side, due to current, caused it to become caught on other anchor cables or bows of boats. In each case skiff was swamped, and crew saved themselves by clinging to pontons or bridge. In such cases it would have been impossible to have cast these anchors with ponton boats and prevented same from becoming lodged above the bridge.

Crews had to be continually cautioned about keeping the bow of the boat upstream or approximately so. They almost invariably lost control of the boats when they pointed them downstream. The current would carry them 100 yards while they were turning around. With properly trained crews, rafts were handled as

easily as single boats. They are, of course, much more stable than single boats, and rendered the work less dangerous.

IV. ANCHORS.

System of anchorage used was as follows:

Several specially constructed anchors were made from railroad and old scrap iron—weight about 500 pounds. To these a spar float consisting of a 12 by 12 inch timber, about 14 feet long, was attached by a ½-inch steel cable 100 feet long. Other anchors were made by attaching three heavy ponton anchors tandem-fashion about 15 ft. apart, using ½-in. steel cable and with floats attached as above.

Fig. 2. Draw open; close view.

Anchors were attached to the cable by using short lengths of cable and wire rope clamps. They were clamped to the cable, and not tied to it. This method was used so that, in pulling up tandem anchors by the winch, the anchors could be detached from the cable as they came up, without removing the cable from the winches and lessening the pull on the following anchors. This was found to work satisfactorily. The anchors were placed tandem fashion and considerable distance apart, so that each one could get a good grip on the bottom without interfering with the others.

Each float had a loop of steel cable through its downstream end. Each ponton boat was provided with an extra Manila cable, having

a large hook in the end, and as the boat dropped down alongside of the float no special difficulty was experienced in hooking the ponton cable to the loop on the float.

These special anchors were cast in such position that the floats swung about 100 feet upstream from the bridge, and were spaced about one float to every three boats.

Number of anchors required:

Experiment showed that, in general, there was no difficulty in holding single boats and rafts of two boats with a single heavy ponton anchor for a short time. Only two or three instances occurred in which a single anchor was dragged by a raft.

Additional anchors were placed as a factor of safety against shifting bottom and increased resistance, due to drift or weight of bridge. The average was about two anchors per boat upstream, and one anchor to each six boats downstream.

Downstream anchors are necessary in swift currents to keep the bridge in proper alignment, prevent oscillation and to counteract the effect of strong winds upstream.

It had been a matter of current belief that the anchors would travel downstream with the shifting bottom, but this was proven not to be the case. The downstream travel after the anchor was once well set was inappreciable, even where the depth of the river increased as much as 12 feet, due to scouring after the anchor had been placed.

Casting anchors:

The specially constructed anchors and most of the triple anchors were cast from two boat rafts. A few of the triple anchors were cast from single boats, but this proved rather inconvenient and somewhat dangerous, due to the amount of cable and possibility of its getting tangled up.

On account of the nature of the bank at the first bridge crossing, it was found practicable to tow the rafts upstream with a team of mules; the vertical bank, with level shelf on top, was a natural tow path.

The crew of the rafts consisted of about twelve oarsmen and several extra men to work the winch and to handle anchors.

Single anchors were first cast at a suitable distance above the bridge, and the rafts were let down on these anchors to the proper place. The triple anchors were then thrown overboard, effort being made to get the three single anchors in line in the direction of the current, and properly placed. By pulling upstream, slack-

ening the cables and rowing down hard with the current and bringing up short on the steel cables, the triple anchors were stretched out and each one given a grip on the bottom. The raft was then pulled upstream over its own anchor and the single anchor pulled up by hand or by winch when necessary. Single anchors could generally be pulled up by six or eight men if the anchors had not been in over an hour or two or had not buried themselves. In the latter case it was necessary to use the winch.

This was an ordinary horizontal four-man affair lashed to the balk near the center of the raft.

It was found necessary to place the winch so as to get a vertical

Fig. 3. Note system of upstream anchors.

pull from a point about 2 or 3 feet forward of the center of the raft. This permitted the use of practically all the buoyant power of the raft and also allowed the raft to swing free with the current, at the same time keeping the bow upstream.

By anchoring a considerable distance upstream and using long cables, it was practicable to cast two sets of triple anchors from the same raft by utilizing the effect of the current to swing the raft sideways.

Single anchors were easily cast in the swift current from single boats by using care in letting the boats drop downstream, stern foremost, and keeping the boats under control of the oars.

Sometimes two separate anchors were cast from one boat by first casting one anchor well upstream and, after pulling back upstream on its cable, casting the other anchor from the opposite side of the boat.

Downstream anchors were cast by towing boats out along downstream side of bridge, letting the boats downstream to cast the anchors, and then pulling them back to proper position in the bridge.

As before stated, about one downstream anchor was required to every sixth boat. In one case the downstream anchor cable was left fastened to the draw span, which had been removed from its place and attached to the shore end of the bridge. This left the last ten or twelve boats of the main bridge without a downstream cable. This part of the bridge had been built with extended intervals, and consequently was not very stiff. A strong wind from downstream (estimated at 20 miles per hour) carried this portion of the bridge upstream past the floats and anchors until some of the anchor cables were pulling downstream. The movement upstream was about 50 feet, and made a considerable bend in the bridge, which moved up and down stream with the wind. The current at this place was at least 4 miles per hour downstream. It was readily prevented by a single downstream anchor, showing that the effects of a 4-mile current and a 20-mile wind are approximately the same.

At certain places, both up and down stream, anchors were required for each boat. Shoals formed in a very few minutes, and the current would change from 4 miles downstream to 2 miles upstream in the same time.

V. CONSTRUCTION OF BRIDGES.

The first bridge was 934 feet long and was placed across the main channel of the Missouri River. Fifteen canvas and thirty-two wooden boats were used. The bridge was constructed by a detail of fifty-four men of Co. E, Engineers, and thirty men from Co. M, Engineers, in eight and one-half hours. All boats were in the water, and the reinforcing anchors had been already placed. This bridge was dismantled in four hours, leaving the boats in the water.

The second bridge was 1,410 feet long, including a pile approach 128 feet long built by Co. G, Engineers. Twenty-four canvas boats, one steel section boat, five trestles and thirty-six wooden

Fig. 4. Shows canvas ponton boats and system of anchors.

boats were used. About fifteen bays were constructed with extended intervals. The bridge was constructed by Co. E, Engineers (average number of men at drill, about 80), in fourteen hours, with the assistance of details from remainder of battalion for about two hours.

The usual working details were made, except two extra cable men and an extra detail to follow along and tighten anchor cables and bail canvas boats where necessary.

Bridges were constructed of successive pontons, as being the only practicable method. Rafts of chess and balk were towed out along the downstream side and lashed to the bridge at the near edge of the main current. This saved a carrying distance of some 600 feet.

The canvas boats were used near each shore where the current was moderate. At the longer bridge it was necessary to use all boats in Engineer Depot, with five Birago trestles additional. The trestles were used to cross a shoal about 400 feet out from shore. This shoal was formed of Missouri mud, covered by a layer of well-packed sand about 1 foot thick. Depth of water was 6 inches to 1 foot. Single balk were lashed just above the trestle shoes from one leg to another across direction of bridge. This developed sufficient bearing power, so that there was no appreciable sinking of the bridge in carrying material back and forth. At one time I saw one trestle supporting the weight of twenty-two men. By lashing more balk to the trestle legs, the bearing power could have been increased five or six times. I believe that at this particular place ordinary army loads could have been taken over the bridge by this method, although my own weight would force a boat hook through the layer of sand and out of sight into the mud below.

This shoal was out of the water about half the time. Due to shifting of the main current after trestles were placed, the shoal was partly washed out, and it was necessary to replace three of the five trestles with canvas boats.

There were several canvas boats in place beyond the trestles, and when the current shifted these boats were left in rather swift water, with current estimated at 7 miles per hour. No difficulty was experienced, however. Heavy anchors had been used for these boats, and several extra anchors were placed by using skiffs as before mentioned. All anchor cables were transferred from the ponton transoms to the floor of the bridge itself and were tied around the upstream balk and side rail. This permitted the can-

vas boats to ride freely without dipping their bows, and it also forestalled the breaking or pulling out of transoms. (See fig. 4.)

No special difficulty was experienced in the construction of the bridge as such. In shoving off each boat, it made matters much easier to shove off the bow first and then let the current do the rest. When the stern was shoved off before the bow it was very difficult to get the bow out, and it had to be held out while being lashed. This, however, was a special case, due to the oblique current towards near shore after shifting of the channel.

About fifteen bays were built at extended intervals. This made the bridge rather weak laterally, but lessened the resistance to the current. Lack of equipment made this expedient necessary.

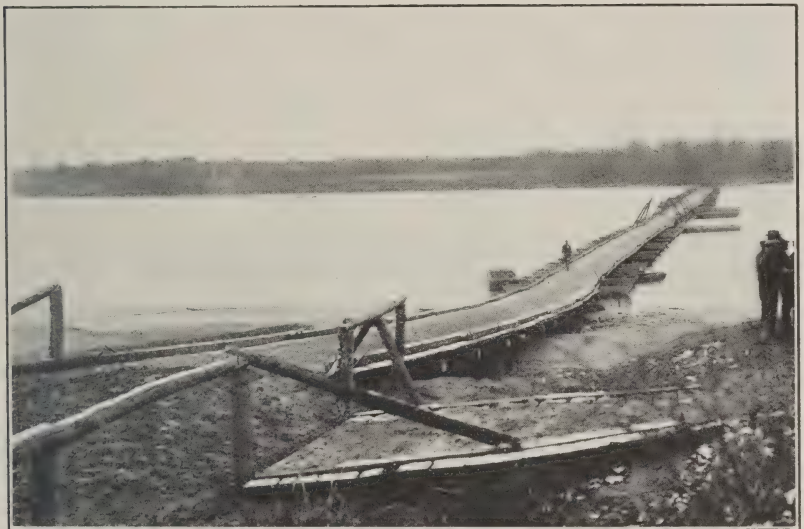


Fig. 5. Bridge, 1,410 feet long, with trestle approach.

A draw span of two bays was built in the bridge and operated. It was not dropped downstream and swung around on account of the strength of the current, but was merely dropped down and towed along the bridge.

Reinforcing triple anchors were used for the boats adjacent to the draw.

All canvas boats were towed along downstream side of the bridge and were never permitted to become detached. Anchors were cast and pulled up from wooden boats.

The most difficult part of the bridge was placed in a downpour of rain and sleet.

The tendency to underestimate distance from head of bridge to far shore must not be forgotten. If the bridge is being built for actual service, it is a safe plan to bring up several extra boats and the corresponding material. This tendency to underestimate distance seems to be nearly universal. I have seen officers make such mistakes as ordering up four more boats when seven were required. I saw another officer at one time, while a regiment of cavalry and two batteries were being held up during a march in maneuvers, make the mistake of ordering no more canvas boats put together and brought up when the distance from last boat to shore proved to be 2 feet greater than the balk would span. This caused a clear delay of nearly fifteen minutes. Other troops become very critical while they are being held up and awaiting completion of a bridge.

It should also be remembered that seven balk should always be used for spans of more than 20 feet.

Flying ferry:

A flying ferry was built at the second bridge, and proved of great assistance throughout the work. The anchor consisted of a set of triple anchors. The cable was held entirely out of the water by using wooden pontons as floats and having the cable passed over a tripod built of round timber lashed in position near the bow of each boat. This permitted the boats to pivot and swing with the current, so that they offered but little resistance to the current. Using a 600-foot cable and a two-boat raft as ferry, we got a swing of about 800 feet, or about the width of swiftest water. The cable was of sufficient length to permit the ferry to swing along the line of anchors. This ferry raft was also equipped with a winch.

An attempt was made to use the three sections of a steel boat as floats, but the middle section was swamped at the first attempt. The other two sections were placed together and used as a boat in the bridge.

VI. MAINTENANCE OF BRIDGES.

The first bridge was left in place two nights and one day; the second, three days and nights. The guard consisted of one officer, three non-commissioned officers, and twelve men. Their duties were such as may have been expected. It was endeavored to keep about equal tension on all anchor cables. The principal trouble came from drift. Two large trees had to be cut up and passed under the first bridge, and three large trees under the second

bridge. All of these came at night. One tree covered five boats in the main channel. It was 3 feet in diameter at the butt. By using extra trestle chains and with balk as levers it was lifted partly out of the water and cut up with axes and saws, and then passed under the bridge. The effect of such work on a dark night with several thousand dollars worth of bridge equipment at stake can readily be imagined. No soldier who served his tour on this bridge will ever forget some of the duties of a sentinel.

VII. DISMANTLING BRIDGES.

No special difficulty was experienced in dismantling bridges, other than in getting up the anchors. Due to shifting bottom, many

Fig. 6. Building portion of bridge with canvas boats.

anchors cast in 18 feet of water were found buried in 12 feet of hard sand bottom, and could not be pulled up. Winches on rafts as above described, were used. The best results were obtained by gradually straining the cables to their limit and keeping up this tension as the anchor became loosened and came up through the sand. A sudden pull only succeeded in breaking the cables.

No difficulty was experienced in getting up anchors that had not been in but a few hours, or had not become buried deep in sand bottom.

VIII. EQUIPMENT.

The action of the reserve and advance guard equipage under stress was all that could have been desired.

Both bridges could readily have been constructed without any extra equipment except cable and anchors. Personally, I can not agree with those people that say our ponton equipment is behind the times, and cite as evidence that it is just the same as used during the Civil War. To my mind, its continued and satisfactory use during those campaigns is the very best argument in its favor. It should be considered also that no extensive improvements have been suggested. There is plenty of suitable timber left, and a reasonable effort would obtain it.

For poor timber, I believe that all boats should be built as designed by Maj. Lytle Brown, Corps of Engineers, and built at the Engineer shops at Fort Leavenworth. This boat was built on the same lines as other boats, except that for the sides and bottom two layers of ½-inch plank were used with a single thickness of heavy canvas between. This kind of boat does not require any caulking, and with poor timber the knots in one board can be so placed that a sound portion of the other board will be underneath. I believe it would be still better to reduce the width of these strips one-half and give the canvas a coating of tar. Reducing the width of the strips one-half would lessen the width of the cracks due to shrinkage. A boat, as designed by Major Brown, was in use the entire time, during which there was no leakage.

It is undoubtedly the best boat in the equipment.

It is regretted that time was not available for trying out the different forms of trestles on hand in the Engineer Depot.

IX. WOODEN *v*. STEEL BOATS.

There is no question but that steel boats can be designed, both solid and sectional, that will give good service under nearly all conditions. The one-section boat and the three-section boat now at Ft. Leavenworth are usable. There is no appreciable difference in the handling of these steel boats and the wooden boats, and no appreciable difference in weight. The steel boats lack stiffness, but this is no great disadvantage, and can be remedied by strengthening the gunwales either with wood or steel. I believe, however, that the principal argument in favor of the steel boat is that after a little experimenting, specifications can be drawn up so that in emergency these specifications could be sent to steel mills and boats constructed by the hundred while our depots were building one wooden boat. Steel boats rust badly, and as now constructed I do not believe their life would be over five years. The principal, and to my mind the

determining, argument in favor of the wooden boat is that it will always float. I should hesitate to ferry troops across a bad river in a steel boat. Men can cling to a wooden boat where there would be no hope with a steel boat. Considerable difficulty would be experienced in raising a sunken steel boat. It is possible that artillery fire could put steel boats beyond immediate repair, and might sink the bridge. Troops could cross a bridge of wooden boats, even if all boats were filled with water. Work on the Missouri River has created in me a strong inclination to remain always near something that will float. This applies to balk as well as boats. Had we been using steel boats during the construction of these bridges, two of

Fig. 7. Bridge completed and in use; note draw span just being closed.

them would have been lost, as was the middle section of another. I should like to see ponton boats built of wood as long as any wood lasts. In time of emergency, if wooden boats can not be furnished rapidly enough, then, of course, steel boats have their place. In designing steel boats, we need not be too particular. Almost any boat along the general lines of the wooden boat will do. If they are at the right place at the right time, there will be no difficulty in using them.

Additional equipment is recommended as follows:

One 20-horsepower gasoline launch with winch attachment for

each ponton train. Also, one four-man force pump with hose attachment for water-jet for each ponton train.

X. TRAINING FOR BRIDGE WORK.

Very little training is necessary for the company as a whole. I do not believe that a company need be detailed permanently as a ponton company. After seven days of rowing around, Co. E, Engineers, built the first bridge across the Missouri River. The only ponton work before this, since 1908, consisted of a few days of ponton work in the Philippine Islands in 1910. If enlisted men are to act as teamsters, they should have considerable training. Otherwise, nearly all the training should be given to the officers and a few of the non-commissioned officers. The actual construction of a bridge is only a minor detail.

The important things to be considered are: Being able to get the equipment through a campaign in good shape; to get the equipment at the right place at the right time; to select the best place of crossing tactically and physically; to determine accurately the material required or makeshifts necessary, and to be able to get it started promptly.

Sometimes it takes longer to build the approaches than to build the entire bridge, and again, the first trestle will cause a long delay. All young Engineer officers should receive training in handling the equipment under many different conditions, and not base their judgment on work in still water with fixed abutment sills and known distance. I believe that most complaints about our equipment, and statements that it is out of date, come from lack of practice in its use.

The Australian Militia System*

BY

Capt. W. G. CAPLES
Corps of Engineers

In December, 1909, Australia adopted universal compulsory military service, but modified the European system to meet the needs of a country which relies upon the militia system. While we are contending with the problem of a force for national defense and talking of forces and organizations, that we know shall be needed but have not, the example of Australia is well worth consideration. It is a well established principle of modern armies that voluntary enlistment is a failure and the national defense can be secured only by compulsory service. An equally well established principle is that the service must be universal else the fate of a nation is decided by the fate of a very small share of its potential strength, and disaster is foolishly courted.

Under the Australian law all male inhabitants, resident for six months in Australia, are registered for military service upon reaching fourteen years of age. Only those who are British subjects receive training.

Training commences with the school children. All boys between the ages of 12 and 14 years are listed as Junior Cadets and receive annually 90 hours training, the object of which is to develop them physically and prepare them for the military service which follows. Two years of this training are required. Many of those who train will fail to pass the doctor when examined for military service, but the cost of the training is not lost for it must have its effect in improving the race. Since the government lacked authority to compel the school teachers to give the training, the latter were exempted from all other military duty if they did give the training. This exemption from military duty has worked satisfactorily. In the United States, the Congress has power to compel this training to

*With comments on its application to our country. (From data in the *Commonwealth Military Journal* for July, 1913.)

be given, under its general authority to raise armies, but some concessions to those who give the training would help to insure its efficient conduct.

Under the Australian law the ninety hours' annual training is distributed as follows:

(a) Physical training, not less than fifteen minutes per day, and any two of the following subjects—

(b) Marching drill (practically infantry squad drill).

(c) Miniature rifle shooting.

(d) Swimming.

(e) Running exercises in organized games.

(f) First aid.

Standards are set for each and inspections are made. The object is to produce collective efficiency, so that the mere expert may help the others and avoid specialization in show pupils.

No uniform is worn by Junior Cadets, but a certain proportion of the teachers are granted honorary commissions. The government allows $1.00 annually for each pupil to the schools to provide for the maintenance of equipment, cost of ammunition, and other necessary expenses. Miniature rifles are loaned to the school if desired.

The same system could be applied to advantage in our schools, while the expense could be met by a small extension of the income tax.

When a boy has passed through his service as a Junior Cadet he is passed to the grade of Senior Cadet, where the instruction now becomes military. The boy is registered at the beginning of the year in which his fourteenth birthday comes. Registration papers are obtained at all postoffices and the boy now comes under the jurisdiction of the Area Officer, to whom the registration papers must be sent. The Area Officer arranges for the medical examination, has all boys who pass measured for uniforms, and, in localities where naval training is conducted, sees that the naval authorities have facilities to select their annual requirements. The pick of the boys are given to the navy, since the navy is recognized as the first line of defense. The navy is manned by a permanent personnel, but three thousand boys from port towns are trained as Senior Cadets and then pass into the Militia Naval Reserve for a period of seven years' training. Australia's conditions are the same as our own, and her system is peculiarly applicable to our requirements. The training of boys in this manner will go a long way toward obtaining

a merchant marine and stopping the drain now imposed upon our resources by shipping in foreign vessels.

Some parents object to the registration just as some people object to compliance with any law, but, as a general rule, parents are only too glad for the boys to get a little discipline and the boys equally glad to get a uniform and a gun. About 7.5 per cent are rejected after medical examination.

The Senior Cadet service covers four years and is intended for boys from 14 to 18 years of age. The annual training includes four whole days, twelve half-days, and twenty-four night drills (quarter days), aggregating 16 days.

A boy passes to the grade of Senior Cadet on July 1 in the year when he becomes 14 years of age, is given his uniform and record book, assigned to a company, and allotted his arms and ammunition. He becomes subject to military discipline and is made to recognize that he forms part of his country's military system.

The Senior Cadet companies consist of 3 officers, 5 sergeants, 4 corporals, and 108 privates, but may be of less strength in the smaller localities. All the companies in a battalion area form a battalion, but the training is essentially company—the battalion appearing only for occasions of ceremony. Senior Cadets attending school are formed into companies in their schools. The record book contains all the rules and is intended to cover a boy's military history until he is 26 years old. The book informs the boy that he must have eleven proficiencies recorded before his military service ceases, and advises him of the rewards for merit and the results of failure.

The minimum number of drills annually required by law are: 4 whole day drills, 4 hours each; 12 half day drills, 2 hours each; 24 night drills, 1 hour each. Drills are not limited to the minimum duration, and many additional voluntary drills and parades are provided. If a cadet misses a drill, without permission, he must attend two extra drills as punishment.

The training is limited to physical drill, company, and a little battalion drill, company field-training, and target practice at bulls-eye and figure-targets up to 400 yards. At the close of the training year, companies are inspected and those members who fail to be declared proficient are required by law to serve an extra year. Not until a man has been declared proficient at eleven annual inspections does his active service cease. An incentive like this would work wonders with our militia.

The annual allowance is \$100 for each locality plus \$0.50 for

each Senior Cadet, plus 150 rounds of ball ammunition for each boy. Extra ammunition may be purchased. Each officer receives from $15 to $35 to cover his expenses. Arms and equipments are kept at company storehouses. The uniforms, consisting of campaign hat, flannel shirt, breeches, and leggins, are supplied by the government. If the system were applied to our country, shoes issued at cost would have to be supplied also. Officers receive the same uniform and an additional allowance at public expense.

Training is encouraged by competitions of teams of 1 officer, 2 sergeants, 1 trumpeter, and 40 rank and file from different companies, in the whole scheme of instruction, including shooting, physical drill, close and extended order, and ceremonies. Each company sends a team to battalion competition. The arrangements may be made by the military authorities, but local citizens are encouraged to help make arrangements and conduct the competition. All athletic games and sports are encouraged. The best team from each battalion takes part in the brigade competition, and the best team from each brigade in the district competition. The winning team of the district is awarded a silver medal for each man and sent to the commonwealth competition, where the winning team receives gold medals. The government pays the cost of the main competition, which is considerable, but the extra efficiency gained is said to be well worth the expense. The expense of battalion competitions and athletic games is covered by charging admission.

At the close of the training year in which the Senior Cadet becomes 18 years of age, he is brought before the medical examiner of the area and examined. Here he is rated as:

(1) Fit; height, 5 feet 4 inches; chest, expanded, 33 inches; eyesight, normal.

(2). Conditionally fit; 5 feet 3 inches; chest, expanded, 32 inches; eyesight below normal.

(3). Temporarily unfit.

(4). Unfit.

(5). Not substantially of European origin and descent; or having religious scruples against service.

Class (1) passes to the militia. Class (2) does the same if there are vacancies, otherwise are retained for a year in the Senior Cadets. Class (3) is retained a year in the Senior Cadets. Classes (4) and (5) are exempted, but men of class (5) physically fit are allotted to non-combatant duties instead of being wholly exempted.

About 16 per cent are rejected annually after the physical examination.

The following are temporarily exempt:

(a) Those living more than 5 miles from training places, or in exempt districts where the population is so sparse that training can not be carried on.

(b) Schoolmasters in charge of cadets.

(c) Members of the regular forces.

(d) Theological students.

(e) Those temporarily unfit as result of medical examination.

There are no absolute exemptions, except for physical unfitness. The Australian proposes to have a white man's army and excludes all others in computing his fighting forces. Out of a white population of about 4,500,000, Australia will have 100,000 Senior Cadets between the ages of 14 and 18 years, and 128,000 militia between the ages of 18 and 25 years. If the same system were applied to the United States, we should have about 2,000,000 Senior Cadets and 2,560,000 Militia. Even though the forces might be only militia, they would be entirely too large to encourage any invasion of our territory.

Under instructions from the district commanders, the brigade majors (adjutants in our army), assisted by the area officers, allot recruits to the various arms according to the following minimum standards:

Arm.	Height.	Chest, expanded.	Eyesight.
		Inches.	
Garrison Artillery	5′ 6″	34	Normal.
Medical Corps	5′ 5″	33	Normal.
Engineers, Field Artillery, Cavalry, Infantry	5′ 4″	33	Normal.
Army Service Corps	5′ 4″	33	Below normal.
Drivers, all arms, 1st line transport	5′ 3″	32	Normal.
Ordnance Corps	5′ 5″	33	Below normal.
All other corps, drivers of 2d line transport of all arms, men allotted to clerical duties	5′ 3″	32	Below normal.

Men are assigned to cavalry only if able to furnish a horse of suitable type. The allotment is completed in time for transfer on July 1 (midwinter).

The annual training is twenty-five days for the Navy, Engineers, and Artillery. Of this period, for the latter two, seventeen days must be in camp. For all other arms the training is sixteen whole

days, of which at least eight must be in camp. The period of service is seven years for all arms or until eleven proficiency credits (including those as Senior Cadets) have been received at the annual inspections. The ages for service are from 18 to 26 years.

Previous to the law compelling service and efficiency, Australia had the same militia system and the same lack of efficiency and other troubles that we have with our militia. It must not be overlooked that under the new law the militia is national, not state. Our state system is the greatest defect with our militia system.

The militia training is carried out along the same lines as the training of regular troops. Target practice takes place on Saturday afternoon, each locality having a target-range near it, although Australia has cities larger than St. Louis and Boston. Cavalry supply their own horses or are transferred to dismounted branches. Field artillery are supplied with horses by the government. When not needed by the field batteries, these horses are available for work of schools of instruction or other mounted units. For other arms, horses are hired as required. Australia is rich in horses, having one to each two white inhabitants, but it is proposed to improve the breed of horses and possibly establish a government breeding station.

For the encouragement of training, competition is carried out in all arms as with the Senior Cadets. How much better is this than in our army, where competitions are limited to target-practice and specialization is encouraged! The pay is \$0.75 per whole day for each soldier (\$12 annually of other arms, and \$18.75 annually for Engineers and Field Artillery) the first year and \$1.00 per whole day in succeeding years (\$16.00 annually for other arms, \$25.00 annually for Engineers and Field Artillery).

Officers and sergeants receive higher pay. A sergeant gets \$2.50 per whole day, a lieutenant \$3.75 per whole day, and other grades are paid in proportion. The maximum annual pay is for the minimum number of whole drill days. In certain cases, an allowance is made for the support of families during the absence of the head on military duty.

Militia are subject to the same rules as regulars, but the scale of punishment is lower and serious offences are left for trial by civil courts.

No liquor is allowed to militiamen on duty, and officers are prohibited from having it at places where militia are training; the fine

for supplying it to a militiaman is $50. A similar law would find good use on our statute books.

Religious scruples are carefully respected, and basphemous and obscene language, indecent and immoral conversation are prohibited; and the rule is reported to be enforced.

Clothing issues are regulated on a scale which will keep each soldier supplied with 1 campaign hat and ornaments; 1 flannel shirt; 2 pair of breeches; 1 pair of leggins; 2 pair of shoes; 1 cap; 1 overcoat; 1 kit bag. Officers and certain non-commissioned officers receive a cloth coat and trousers in addition, and officers certain other articles for full dress.

The garments are of a universal pattern, the arms being distinguished by the color of the hat band.

All promotions are made from the ranks and based upon competitive examination of a practical character and are, as far as possible, oral, the only written work being such as the candidate would have to carry out in the rank for which he is being examined. This system holds good from corporal to captain. Above captain, the practical examination counts two-thirds. All promotions are competitive and from the next lower grade. All officers are required to serve twelve years as an officer.

Schools of instruction are held at many places at which continuous training is given from one to four weeks. There is no extra pay for attendance, but quarters and messing are provided free. Here the ambitious prepare for promotion, which is said to prove sufficient incentive to secure attendance. The necessary books are provided free.

A maximum length of service is provided in each grade and, if an officer fails to be promoted within that time, he is transferred to an unattached list. These periods are five years as lieutenant, four years as captain, three years as major, and three years as lieutenant-colonel. This system gives a large reserve of somewhat trained officers. It is computed that in each battalion there will be annually two vacancies for captain and six lieutenants to compete. Two will be promoted and retained, two promoted and unattached; what becomes of the other two is not stated. Unattached officers complete their service as officers by serving sixteen days annually on staff or special duty, at schools of instruction, or with Senior Cadet units. Their pay is the same as if on active list.

One of the most remarkable features is the small expense at-

tached. The budget for 1912-1913 carried the following items:

Central administration	$375,000
Royal Military College	191,565
Central schools of instruction, staff instructions, physical training, aviation, and target practice	143,910
Inspection of stores and manufacturing establishments	480,285
District commands, staffs	127,500
Instructional staff	672,250
Field artillery	244,330
Garrison artillery	599,565
Engineers	260,875
Other permanent services, civil and military	415,350
Militia, all arms	2,393,875
Volunteers (aviation, automobile corps, nurses)	3,040
Cadet, Junior and Senior, uniform, allowances, etc.	1,347,750
Rifle clubs and associations	183,515
Ammunition	861,050
Camps and schools of instruction	415,000
Military equipment, stores, and reserve ammunition	1,953,000
Fixed armament	603,250
Works and construction, rifle ranges, rents, and maintenance	1,503,760
Interest on lands and buildings, resumed	605,995
Miscellaneous services, etc.	479,485
Total	$14,862,350

Australia had then to pay about $15,000,000 to maintain a force of 2,719 regulars, 43,731 militia actually training, and a growing reserve of 89,000 Senior Cadets.

The Australian system is of peculiar adaptability to the United States with its absence of any aggressive intention as regards other nations, but also with an absence of any adequate means of national defense. By making our militia national instead of state, compelling all male citizens to undergo training (starting in the school room and extending into their young manhood) and then paying the bills for what we want, we could develop at moderate expense a force which would make our borders safe from attack.

Deflection of Unstiffened Suspension Bridges

BY

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It is well known that a suspension bridge is subject to deflection under concentrated or unsymmetrical loads. To prevent excessive deflections, the more elaborate suspension bridges are provided with stiffening trusses properly proportioned to distribute these loads. In less important bridges these trusses are sometimes omitted, or often consist at most of a braced railing that is entirely inadequate to resist any considerable stress, and that must consequently yield to the deflections. Unstiffened, or inadequately stiffened, bridges are most frequently met with in temporary or pioneer construction, in the design of which the type is attractive, since it requires no subaqueous foundations and no falsework, and is easily erected by unskilled labor. Many such bridges are most satisfactory under the loads to which they are subjected, but occasionally a bridge is found to be unsatisfactory on account of the vibrations set up by excessive deflection. It is the purpose of this article to derive formulæ for determining these deflections.

It is obvious that the deflection of a suspension bridge is due to change in the curve of the cable. The deflection may be divided into: (a) The inelastic deflection, due to concentrated or unsymmetrical loads, the length of cable remaining constant. (b) The deflections due to the elongation of the cable under load, and under change of temperature.

As suspension bridges are usually constructed, the second is relatively unimportant. We shall, therefore, consider for the present the inelastic deflections only.

CURVE OF THE CABLE UNDER UNIFORM LOAD.

Consider a bridge (Fig. 1) whose span is l. Let f be the sag of the cable at the mid point, and W the total weight of the load, assumed uniformly distributed over the bridge. Let H_0 be the

horizontal component of the tension of the cables. It is easily shown that H_0 is constant. Let y be the ordinate of the curve of the cable at any point M, and x the horizontal distance from the left support to this point.

Equating the moments at M, we have:

$$H_0 y = \frac{1}{2} W x - \frac{W}{l} x \cdot \frac{1}{2} x \quad \text{or} \quad y = \frac{1}{2} \frac{W}{H_0} \left(x - \frac{x^2}{l} \right).$$

$$\text{When } x = \frac{1}{2} l,\ y = f;\ \text{whence } H_0 = \frac{1}{8} W \frac{l}{f} \qquad (1)$$

The equation of the curve of the cable, under uniform load, is therefore:

$$y = 4f \left(\frac{x}{l} - \frac{x^2}{l^2} \right) \qquad (2)$$

A parabola with its vertex at the mid point.

CURVE OF THE CABLE UNDER A CONCENTRATED LOAD.

Let, now, a concentrated load of weight P be placed on the bridge at a distance kl from the left support.

The vertical reactions are:

$$\text{Left support}: R_1 = \frac{1}{2} W + (1-k) P$$

$$\text{Right support}: R_2 = \frac{1}{2} W + k P$$

Let H be the constant horizontal component of the cable tension.

For the equation of the curve of the cable, A to K, we have, equating moments as before:

$$y_1 = \frac{\frac{1}{2} W + (1-k) P}{H} x - \frac{1}{2} \frac{W}{H} \frac{x^2}{l} \qquad (3)$$

Similarly, the equation of the curve, K to B, is:

$$y_2 = \frac{kP}{H} l + \frac{\frac{1}{2} W - kP}{H} x - \frac{1}{2} \frac{W}{H} \frac{x^2}{l} \qquad (4)$$

Both branches are parabolas, with their axes vertical.

To determine the value of H, we impose the condition, in accordance with our assumption, that the length of the cable remains constant under the addition of the weight P. This length is the sum of the arcs AK and KB.

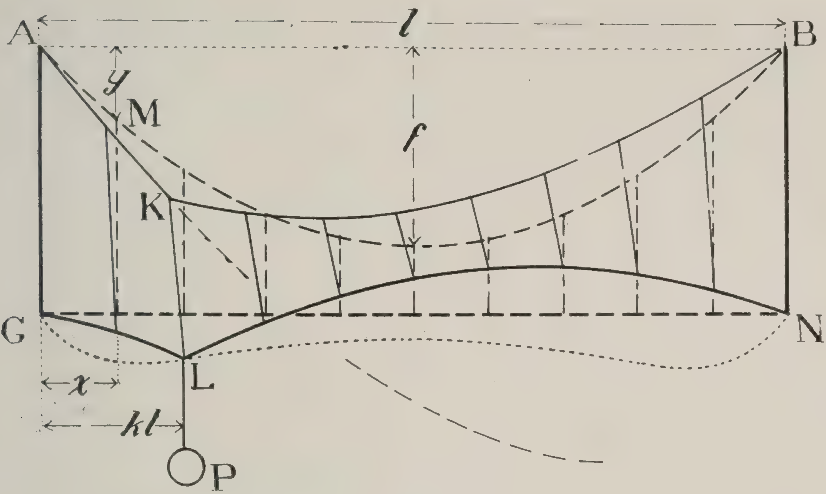

Fig. 1.

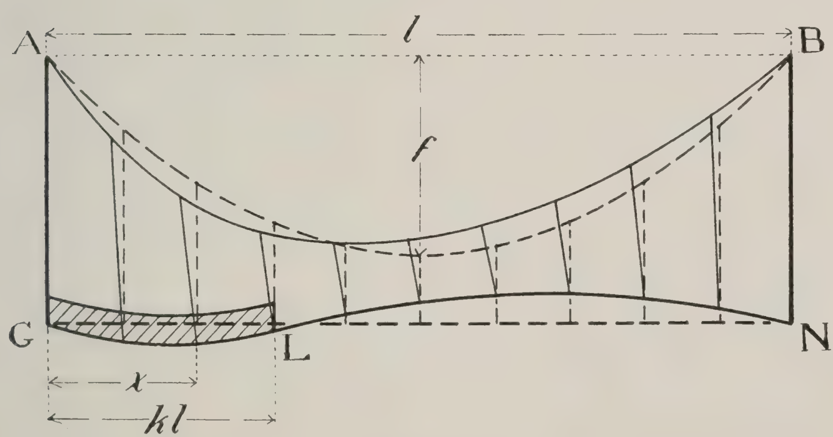

Fig. 2.

To determine the length of the arc AK, place, for shortness:

$$\frac{\frac{1}{2}\mathrm{W} + (1-k)\ \mathrm{P}}{\mathrm{H}} = a \qquad\qquad \frac{\mathrm{W}}{\mathrm{H}l} = b$$

so that equation (3) becomes

$$y_1 = ax - \frac{1}{2}bx^2$$

The length of the arc of this parabola is

$$s_1 = \int_0^{kl} \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\ dx = \int_0^{kl} \sqrt{1 + (a - bx)^2}\ dx.$$

The closed integral is somewhat complicated. Noting that a and b are small when the sag ratio $\frac{f}{l}$ is small, we may expand the radical and neglect terms of higher degree than the second in a and b. We have, then,

$$s_1 = \int_0^{kl} \left[1 + \frac{1}{2}(a - bx)^2\right] dx = kl + \frac{1}{2}a^2kl - \frac{1}{2}ab\,k^2l^2 + \frac{1}{6}b^2k^3l^3$$

Replacing a and b by their values above, we obtain, for the length of the arc AK, after reduction:

$$s_1 = kl\left(1 + \frac{\mathrm{W}^2}{\mathrm{H}^2}\left[\frac{1}{8}(1-k)^2\left(1 + \frac{2\mathrm{P}}{\mathrm{W}}\right)^2 + \frac{1}{24}k^2\right]\right)$$

The length of the arc KB may be found by substituting $(1-k)$ for k in the above, and is:

$$s_2 = (l - kl)\left(1 + \frac{\mathrm{W}^2}{\mathrm{H}^2}\left[\frac{1}{8}k^2\left(1 + \frac{2\mathrm{P}}{\mathrm{W}}\right)^2 + \frac{1}{24}(1-k)^2\right]\right)$$

The total length of AKB is therefore

$$s = s_1 + s_2 = l\left[1 + \frac{1}{2}\frac{\mathrm{W}^2}{\mathrm{H}^2}\left(\frac{\mathrm{P}}{\mathrm{W}} + \frac{\mathrm{P}^2}{\mathrm{W}^2}\right)(k - k^2) + \frac{1}{24}\frac{\mathrm{W}^2}{\mathrm{H}^2}\right]$$

When $\mathrm{P} = 0$, $\mathrm{H} = \mathrm{H}_0 = \frac{1}{8}\mathrm{W}\frac{l}{f}$, and the above expression becomes

$$s = l\left(1 + \frac{8}{3}\frac{f^2}{l^2}\right)$$

the length of the cable under dead load only.

But since the length does not change, these two expressions may be equated, giving:

$$l\left(1+\frac{8}{3}\cdot\frac{f^2}{l^2}\right)=l\left|1+\frac{1}{2}\frac{W^2}{H^2}\left(\frac{P}{W}+\frac{P^2}{W^2}\right)(k-k^2)+\frac{1}{24}\frac{W^2}{H^2}\right|$$

Whence,

$$H=\frac{1}{8}W\frac{l}{f}\left[1+12\left(\frac{P}{W}+\frac{P^2}{W^2}\right)(k-k^2)\right]^{\frac{1}{2}} \qquad (5)$$

This value, substituted in equations (3) and (4), gives the equation of the curve of the cable under the load P.

DEFLECTION OF THE FLOOR UNDER A CONCENTRATED LOAD.

To arrive at comparatively simple formulæ, two assumptions will be made:

(a) That the deflection in the floor is the same as the deflection of the cable at the same vertical section.

(b) That the deflection is proportional to the load.

While these assumptions are but approximately correct, both are on the side of safety. They will be discussed later.

Under these assumptions the deflection of the floor due to a small load dP is found by differentiating equations (3) and (4) with respect to P, and placing P=0 in the result. We obtain thereby, for deflections between G and L (Fig. 1), from equation (3)

$$dy_1=\frac{8f}{W}\left[(1-k)\frac{x}{l}-\frac{4f}{l}\left(\frac{x}{l}-\frac{x^2}{l^2}\right)\frac{dH}{dP_0}\right]dP$$

And from equation (5),

$$\frac{dH}{dP_0}=\frac{3l}{4f}(k-k^2) \qquad (6)$$

Substituting this value, and writing P for dP, in accordance with the second assumption, we have, for deflections between G and L

$$dy_1=8\frac{P}{W}f\left|(1-k)\frac{x}{l}-3(k-k^2)\left(\frac{x}{l}-\frac{x^2}{l^2}\right)\right| \qquad (A)$$

Similarly, deflections between L and N are given by the formula

$$dy_2=8\frac{P}{W}\cdot f\left|k\left(1-\frac{x}{l}\right)-3(k-k^2)\left(\frac{x}{l}-\frac{x^2}{l^2}\right)\right| \qquad (B)$$

These are the formulæ proposed. To obtain the deflection due to any combination of loads, we may sum the deflections at any point

due to the loads taken separately, proper regard being had to sign.

It will be noted that the deflection at any point varies directly as the sag of the cable, and inversely as the dead weight of the bridge. The fact that stability can be secured by using a substantial floor system and taut cables is well known.

DEFLECTIONS UNDER A DISTRIBUTED LOAD.

While it will rarely happen that a distributed live load of uniform weight per unit of length will cover a portion only of the bridge, yet the formulæ for the deflections caused by such loading are of interest. These formulæ are deduced as follows (Fig. 2).

Let p be the weight of the live load, and w the weight of the dead load per unit of length. Let the live load extend a distance kl from the left support. The deflection at any point x will then be the sum of the deflections due to the elementary loads $p\, d(kl)$.

Substituting therefore $lpdk$ for P and lw for W in equation (A) and integrating, we have, for the deflection at any point between G and L

$$dy_1 = 8\frac{p}{w}f\left[\left(k-\frac{k^2}{2}\right)\frac{x}{l}-3\left(\frac{k^2}{2}-\frac{k^3}{3}\right)\left(\frac{x}{l}-\frac{x^2}{l^2}\right)\right]+C$$

When $k=1$, the bridge is uniformly loaded, and $dy_1=0$. From this condition we find that $C=-4\frac{p}{w}f\frac{x^2}{l^2}$. Substituting and reducing, we have

$$dy_1 = 4\frac{p}{w}f(1-k)^2\left[2k\frac{x}{l}-(1+2k)\frac{x^2}{l^2}\right] \tag{7}$$

Similarly, for deflections between L and N

$$dy_2 = 4\frac{p}{w}f\,k^2\left[1-2(2-k)\frac{x}{l}+(3-2k)\frac{x^2}{l^2}\right] \tag{8}$$

Derived from the condition that when $k=0$, $dy_2=0$.

MAXIMUM DEFLECTIONS.

Under a single concentrated load, the greatest positive, or downward deflection evidently occurs at the point of load, $x=kl$. It is easily shown that this deflection is a maximum when the load is at a distance $0.21l$ from a support, and that the resulting deflection is $\frac{2}{3}\frac{P}{W}f$.

The maximum upward, or negative deflection, is found to occur when the load is a distance $0.27l$ from one support, the point of greatest deflection is the same distance from the other support, and the amount of deflection is $-0.35\,\frac{P}{W}f$.

Under a uniformly distributed load, the maximum positive deflection occurs at a point $0.23l$ from a support, when the load extends a distance $0.43l$ from this support, and the amount of the deflection is $0.13\frac{p}{w}f$.

The maximum negative deflection occurs at the same distance $0.23l$ from the support, when the load extends a distance $0.57l$ from the other support, and the amount of the deflection is $-0.13\frac{p}{w}f$.

The maximum deflection, in both these extreme cases, occurs therefore near the quarter point.

DISCUSSION OF RESULTS.

The foregoing formulæ are based on the following approximations:

(a) The value of $\frac{dH}{dP_0}$ (equation 6) is derived from an approximate value of H (equation 5).

(b) The deflection of the floor is assumed as that of the cable at the same vertical section.

(c) The deflection is assumed to be proportional to the load.

(d) The elongation of the cable is neglected.

The mathematical work leading to a discussion of these errors is somewhat tedious and may not be of interest; we shall therefore limit ourselves to the results.

(a) The true value of $\frac{dH}{dP_0}$ (for an inelastic cable) is

$$\frac{dH}{dP_0} = \frac{\sqrt{1+h^2} - \sqrt{1+h^2\,(1-2k)^2}}{h\,\sqrt{1+h^2} - \log_e\left(h + \sqrt{1+h^2}\right)}$$

Where $h = \frac{4f}{l}$.

Assigning to k the value .211 which was seen to give the maximum deflection, we may form the following comparison:

$\frac{f}{l} =$	$\frac{1}{4}$	$\frac{1}{6}$	$\frac{1}{8}$	$\frac{1}{10}$	
$h =$	1	$\frac{2}{3}$	$\frac{1}{2}$	$\frac{2}{5}$	
$\frac{dH}{dP_0}$ (equation 5)	.500	.750	1.000	1.250	$\frac{P}{W}$
$\frac{dH}{dP_0}$ (exact)	.487	.740	.992	1.244	$\frac{P}{W}$

The error from this source is seen to be very small.

(b) In taking the deflection of the floor as that of the cable at the same vertical section, we have assumed that the tops of the suspender rods move vertically under the weight P. But it is evident that these hangers must, in fact, move in such manner that the length of the cable from the point of attachment to the tower remains constant. The vertical component of this displacement will be the deflection of the floor. Instead, therefore, of differentiating equations (3) and (4), regarding x as constant, these equations should be differentiated under the condition that $\int_0^x ds$ is constant. Pursuing this course we obtain, as a correction for the deflection as determined in equation (A), the complicated expression:

$$\delta = -4\frac{P}{W}f\,\frac{\left(1-2\frac{x}{l}\right)}{q\left(\frac{x}{l}\right)}\left\{(1-k)\left[q(0)-q\left(\frac{x}{l}\right)\right]-h\,\frac{q(0)-q(k)}{h\,q(0)-\log_e\left(h+q(0)\right)}\right.$$

$$\left.\cdot\left[\frac{x}{l}q\left(\frac{x}{l}\right)+\frac{1}{2}q(0)-\frac{1}{2}q\left(\frac{x}{l}\right)+\frac{1}{2h}\log_e\frac{h\left(1-2\frac{x}{l}\right)+q\left(\frac{x}{l}\right)}{h+q(0)}\right]\right\}$$

Where $h=\frac{4f}{l}$, and $q(z)=\sqrt{1+h^2(1-2z)^2}$

As an indication of the error of this assumption, we have placed $\frac{x}{l}=k=.211$, and computed the value of the correction:

$\frac{f}{l}$	$\frac{1}{4}$	$\frac{1}{6}$	$\frac{1}{8}$	$\frac{1}{10}$	
dy (Equation A)	.667	.667	.667	.667	$\frac{P}{W}f$
Correction δ	—.207	—.110	—.067	—.044	$\frac{P}{W}f$
	31	18	10	7	

The correction is seen to be on the side of safety, and to be moderate for the sag ratios usually employed.

(*c*) It is evident that as the load increases the curve of the cable changes, and that the deflection is not in fact proportional to the load. Taking this source of error only, the deflection due to concentrated load P will be the difference in the ordinate y and y_1, equations (2) and (3) and in y and y_2, equations (2) and (4) respectively.

Taking the approximate value of H (equation 5), and placing $x=kl=.211l$ (the values giving the maximum deflection under equation A), we have the following results:

$\frac{P}{W}=$	.05	.10	.15	.20	.25	
Deflection, equation A=	.033	.067	.100	.133	.167	f
Deflection, corrected=	.031	.058	.081	.101	.118	f
Error	8	13	19	25	30	

It will be seen that for large loads the formula gives results considerably too great.

An empirical correction may be made by placing W, in equations A and B, equal to the *total* load on the bridge, instead of the dead load only. We then obtain the following results, for $x=kl=.211l$.

Ratio $\frac{\text{Live load}}{\text{Dead load}}=$	.05	.10	.15	.20	.25	
Deflection, equation A=	.032	.061	.087	.111	.133	f
Deflection, corrected=	.031	.058	.081	.101	.118	f
Error	3	5	7	10	12	%

The error is much reduced, and is still on the side of safety.

(d) The deflection due to the elasticity of the cable is dependent on its cross section. Formulæ for this deflection may be found in Merriman and Jacoby, and in other text books. Modifying these

formulæ to conform to the symbols herein employed, we have the approximate expression

$$dy = \frac{3}{4}\frac{l}{f} \cdot e \cdot \left(\frac{x}{l} - \frac{x^2}{l^2}\right)$$

Where e is the elongation of the cables.

The elongation of the cables between the towers is

$$e_s = \frac{3}{4}\frac{\mathrm{P}}{\mathrm{AE}}\frac{l^2}{f}\left(1 + \frac{16}{3}\frac{f^2}{l^2}\right)$$

Where A is the net area of the metal in the cables, and E the modulus of elasticity. To this must be added the elongation of the backstays.

The deflection thereby obtained may be added to that determined by equations (A) and (B). It will be found, in ordinary cases, so small a correction as to be unimportant.

It may be noted, however, that the increased sag due to the weight of the floor is a point which frequently requires attention in the erection of the bridge.

To sum up, it seems justifiable to conclude that the deflection obtained by formulæ A and B, while not accurate, are sufficiently reliable for the purpose for which they are intended.

Claude Crozet*

BY

Maj. Gen. WILLIAM HARDING CARTER
United States Army

When the high speed with which our modern industrial age is rushing forward with the development of a mighty empire begins to slacken, there will come a period of leisure and of more general study as to the men and means by which we have accomplished in a century the normal work of a thousand years. A single invention makes necessary a score of subordinate ones and with the aptly characterized wizards of the air, of electricity, of agriculture, and of every applied science sleeplessly pressing onward, the luxuries of yesterday become the necessities of to-day and our minds refuse longer to be surprised or dismayed at even the most weird and impossible suggestions. When the nation pauses and begins the effort to orient itself after the great onward rush, the men who have helped to upbuild the great Republic will assume their proper place in history and stand out more prominently in the process of sifting the material from the immaterial, the real from the visionary, the stable from the unstable.

However much we may gloss over the facts, there was a period in our history during and following the War of 1812 when the nation builders pondered deeply upon the question of our ability to make good the experiment of creating a workable government out of an aggregation of widely scattered commonwealths with varying views

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*Claude Crozet was never an officer of the Corps of Engineers, but he was Professor of Engineering at West Point when West Point was still considered a school for engineers only. Not only is this true, but he was Professor during the years of greatest development under Colonel Thayer, "The Father of the Military Academy," and during his seven years as professor developed and extended the course in engineering, and wrote the first text-book on descriptive geometry ever used in this country.

Reprinted from the *Journal of the Military Service Institution of the United States*, for July-August, by special permission of both General Carter and the publishers of the *Journal*.

as to their sovereignty and State rights. The glory of New Orleans could not wipe out the disgrace of other fields. The capture and sacking of the capital caused humiliation not easily removed from the minds of the patriotic men to whom the problem of resisting invasion had been committed. To improve the country's defense and to make the assault of our coasts more difficult and dangerous to an enemy, engaged the earnest attention of those responsible for the administration. In seeking assistance from those who had learned the art of war under Napoleon, our nation secured the services of the incomparable Bernard, whose example in seeking service in America was followed by another veteran of France, Captain Claude Crozet, whose career is worthy of a page of our military history. Those were stirring times in Europe and the *élève* of the *Ecole Polytechnique* of to-day becomes the battle-scarred veteran of to-morrow, as our cadets of '61 became the division commanders of '64, and life histories of such types of men will live to fire the blood of youthful endeavor and some day make it as honorable to qualify for the country's defense as for success in the marts of trade.

Capt. Claude Crozet was born at Villefranche, near Lyons, France, January 1, 1790. At 14 years of age he was admitted to the Polytechnic School, Paris; was graduated in 1807 as sub-lieutenant of artillery, and then proceeded to Metz for the special course of instruction given there to officers assigned to engineers and artillery. After two years at this fortress, he joined the headquarters of the Emperor near Vienna just in time to participate in the memorable battle of Wagram, July 5, 1809. During the next two years young Crozet received from the hands of the Emperor the cross of the Legion of Honor, his promotion to the grade of captain and an assignment to the Imperial Corps of Artillery attached to the division of Marshal Ney, then making preparations for the invasion of Russia.

On the retreat from that disastrous campaign, Captain Crozet was taken prisoner but had the good fortune to fall under charge of one of the Russian princes, who became interested in him and sent him with an escort to his own estate in the interior, where he remained under the care of the prince's family for two years, the recipient of most kindly hospitality.

After the Treaty of Paris, in 1814, Captain Crozet returned to France, arriving in Paris on the 20th of August, several months after the departure of Napoleon from Fontainbleau for the island

of Elba. In accordance with the conciliatory policy adopted by the restored government, the "Decoration de Lys" was, by order of the king, conferred upon Captain Crozet, together with restoration to his former rank in the army, but he declined the honor, avowing his desire to engage in civil pursuits, and established himself in Paris, where were gathered a multitude of ex-officers who, out of employment, were speculating upon the next turn of the wheel of fortune.

Captain Crozet was not an unmoved spectator when, on the 20th of March, 1815, Napoleon re-entered Paris and was greeted with enthusiasm by the populace who believed the destiny of their soldier emperor had not yet been accomplished. Captain Crozet resumed his position in the army and joined in the hurried preparations for the great struggle which, on the field of Waterloo, terminated Napoleon's active career. With the downfall of Napoleon Paris was filled once more with unemployed officers and Crozet determined to seek another field of opportunity. He had married Mademoiselle de Camp, and on the 6th of June, 1816, provided with letters from Marquis de Lafayette and other gentlemen of prominence, he embarked for America and arrived in New York about the middle of July. His scientific attainments having become known, and the course at West Point being just then in process of revision, he was appointed Professor of Engineering and entered upon his duties at the Academy February 1, 1817. Under Captain Crozet instruction was first given at West Point in descriptive geometry, analytical trigonometry, differential and integral calculus, civil engineering and the principles of machines. At that time there was no text-book on descriptive geometry in this country and until Captain Crozet's treatise was issued in 1821, instruction in the subject at West Point was entirely oral.

The intense mental application and six years of sedentary classroom work following an active career in the armies of Napoleon so affected the health of Captain Crozet that he resigned the professorship at West Point and entered the service of Virginia as State Engineer, a position affording an opportunity for more outdoor life. He entered upon his duties in 1824 and continued in the service of the State for nearly nine years. Captain Crozet very ably urged a lock and dam system of improvement of the James River from Richmond to Lynchburg, and in 1830 further urged the construction of a railroad thence to the Kanawha River, thus uniting the eastern and western waters.

In 1832 Captain Crozet accepted a position in Louisiana similar to that held in Virginia, but after a year spent amid the swamps and lagoons of that region he became the president of Jefferson College, on the banks of the Mississippi about one hundred miles from New Orleans. The following year he was on the eve of visiting France when he was urged to return to Virginia and resume his position as Engineer of the State. He changed his plans, and in 1837 resumed his work in Virginia. In the meantime Virginia had entered upon internal improvements under a joint stock system whereby she became a partner and manager to the extent of her interest. The duties of the State Engineer then became primarily to observe the management and condition of such works as the State was interested in, and to make such suggestions as he might deem proper. His suggestions and recommendations indicate a masterly if not prophetic mind.

In 1837, Captain Crozet was chosen president of the board of visitors of the Virginia Military Institute just then established, the cadets being substituted for the guard of the State arsenal at Lexington, which had been maintained since 1816 for the preservation of the arms and military equipments of the State. Captain Crozet took an active part in the organization and advancement of the interests of the Institute. In 1845 he undertook the preparation of a series of books on mathematics, but the writing was too confining and, upon completion of one volume, an arithmetic, was abandoned.

In 1849 Captain Crozet was selected to locate and construct the Blue Ridge railroad from Albemarle County through Rockfish Gap to Augusta County, as a State improvement. This proved a very difficult undertaking and involved construction of several tunnels with many complications. The work was completed and turned over to the predecessors of the present Chesapeake and Ohio Company in 1856, and soon after Captain Crozet was invited to Washington by the Secretary of War to assume the position of principal assistant to Capt. M. C. Meigs, Corps of Engineers, in the construction of the aqueduct. Captain Crozet is credited with the planning and construction of the existing aqueduct bridge connecting Georgetown with the Virginia shore near the Arlington estate. He was separated from the aqueduct engineering work in 1859 on account of exhaustion of funds, and returned to Richmond, Va., where he resided until his death in 1864. His character and

code of ethics are observable in his qualified letter of acceptance of employment as assistant engineer of the Washington aqueduct.

RICHMOND, *Nov. 21, '57.*

Capt. M. C. MEIGS,

Dear Sir: When the Secretary sent for me, I had not the least idea of his object, but merely surmised, knowing him to be very friendly to me, that it was for some purpose advantageous to myself. However, I never coveted nor accepted any office unless vacant, or certainly to be vacated. My feelings, in the present instance, are, of course, in accordance with my usual practice; and the more so in this case that I have ever experienced and reciprocated the most friendly feelings on the part of the officers of the Army, and especially those of the Engineer Corps. If, therefore, you think you can by any possibility retain your assistant if I decline, let me know, confidentially, and I will do it. If, on the contrary, a change is decided upon, I would accept for several reasons which I will communicate hereafter, and I must add that I have no doubt that our relations would be satisfactory to both. Having, of course, to supply your place and carry out your own plans, and possessing the ability of understanding them, I do not apprehend the possibility of any difficulty. As regards relative positions, we are all dependent in some way or other, and as an old soldier, I understand the value of discipline, without which no service can be efficiently rendered. I shall await your answer before I come to a decision. In the meantime accept the assurance of my high consideration.

Yours,

C. CROZET.

Captain Crozet was a man eminent in all the professional work undertaken by him and left a singularly deep impression upon those who fell under the spell of his teachings. His engineering work has survived to give silent testimony to his genius and fidelity. His career was not so brilliant, nor were such honors showered upon him as upon General Bernard. Capt. Crozet rather took his modest place among the commonwealth builders of a day and generation when the nation was reaching out and preparing an empire of territory for the great agricultural and industrial development which later drew to our shores millions of liberty-loving Europeans. The work of these two veterans of France should live in the hearts of Americans, for they rendered the State inestimable service when talents such as theirs were sufficiently rare to be in demand throughout the civilized world.

Mississippi River Gagings by Rod Floats

BY

Mr. FREDERICK YANCY PARKER
Assistant Engineer; Member American Society Civil Engineers; Member Western Society of Engineers

The first recorded attempt by the St. Louis engineer office to obtain a river gaging, was made in May, 1872 (report of the Chief of Engineers, U. S. A., for 1873, page 472). From that time until 1895 intermittent gagings were taken, usually with surface and double floats; but at no regular station. During low water periods, 1895 to 1899, gagings were taken wherever a field party happened to be located.

As a comparison of results, under the above conditions, was of questionable value, observations were begun in 1900 at the present low water gaging station, U. S. Engineer Depot, foot of Arsenal Street, St. Louis, Mo., and have been continued to the present time. This site was selected on account of convenience, comparative stability of banks and bottom, and parallelism of stream flow. From 1900 to date, rod floats have been used at this and other stations at all stages of the river with satisfactory results. The gaging hereafter described was taken December 30, 1912. As this was the first of the season and the lines of stream flow unknown, the usual sextant observations, at intersections of floats and range lines, were supplemented by transit readings from the downstream end of the base.

To reduce the observed rod velocity to the mean velocity of stream thread from surface to bottom, Francis formula was used.

$$Vm = Vr\,[1 - .116\,(\sqrt{D} - .1)].$$

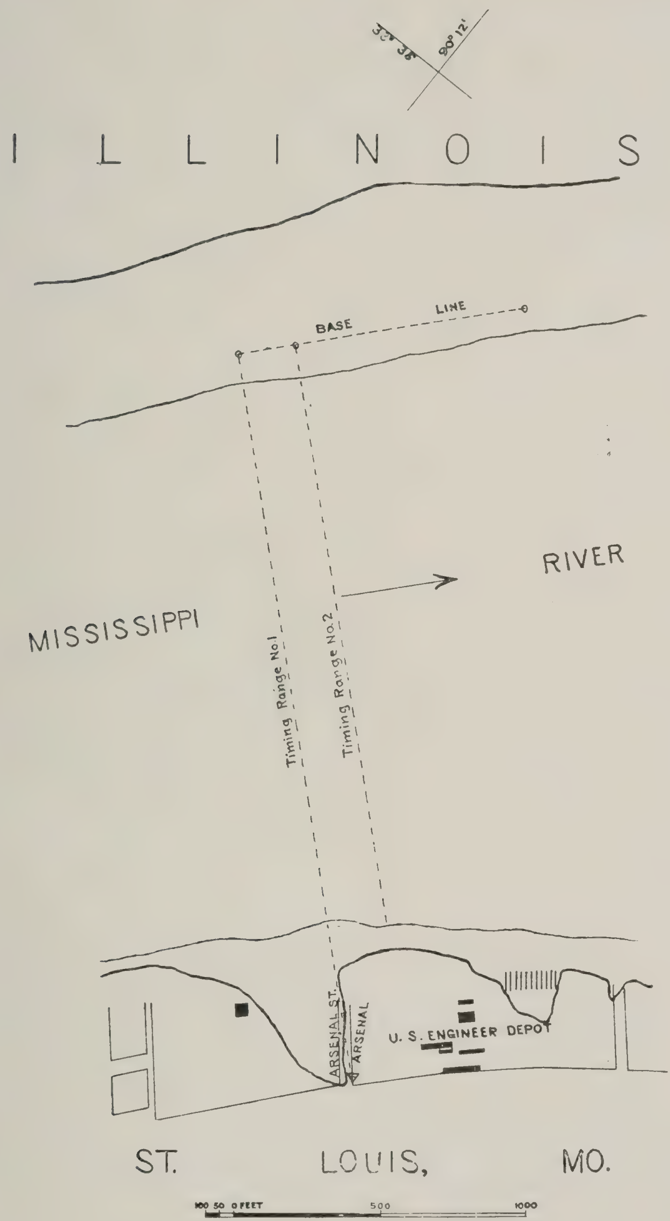
Vr = observed velocity of rod.

Vm = mean velocity in a vertical plane.

$D = \left(\dfrac{\text{depth below bottom of rod}}{\text{depth of stream}}\right)$ or the percentage of stream depth unexplored by the rod.

BASE LINES.

For low water and medium stages, a base line is measured on a



38° 36'
90° 12'
I L L I N O I S
BASE LINE
RIVER
MISSISSIPPI
Timing Range No.1
Timing Range No.2
ARSENAL ST.
ARSENAL
U. S. ENGINEER DEPOT
ST. LOUIS, MO.
100 50 0 FEET 500 1000

Fig. 1.

bar along the Illinois bank. (Fig. 1.) This site is desirable on account of the unobstructed view and freedom from interruption by river traffic; but, as the bar is submerged at the 26-foot stage, St. Louis gage, a station at the southern city limits is used for stages not exceeding 31 feet. The nearest suitable station for flood stages. is Chester, Ill., 75 miles downstream. Until recent years a permanent base line was maintained in the U. S. Engineer Depot grounds, but new buildings have obstructed the view, and shifting shore lines and fill have nullified attempts at reestablishment on the bar. However, the position on the bar can always be relocated, since the range system is hinged on Arsenal (triangulation station)—stone pillar in Engineer Depot yard.

On a line, approximately normal to the stream flow and passing through Arsenal, a point on the bar is selected for the upper limit of the base; from this point 1,000 feet (for ease in computing) normal to the line is measured downstream. Targets are erected at each end of the base, at Arsenal and on the prolongation of the line—Arsenal, upper base limit; this line is used as timing-range No. 1. Parallel to timing-range No. 1 and 200 feet downstream, timing-range No. 2 is located; with timing station on the base line. (Fig. 1.)

After the base line and ranges are laid out, both of the unknown angles of the right-angled triangle—Arsenal, upper base limit, lower base limit—are determined by repetition, and the distance between Arsenal and upper base limit computed. This distance (in the gaging described 2,502 feet) is a function of the upstream base extremity and a constant for all other points of the system.

FLOATING PLANT.

The floating plant consists of a tug, flat boat, and two skiffs. The flat, lashed end-on to the bow of the tug, facilitates storing, launching, and recovering the floats. One skiff, containing two men and equipped with a small anchor and line, is used to buoy each float station until its gaging is completed and the succeeding station located.

The second skiff carries, in addition to the crew of three men, a recorder, observer, and leadsman. The tug is of the usual sternwheel river type and carries a crew of five.

FLOATS.

A set of floats consists of twenty-six pieces: namely, eight extension rods, each 13.5 feet long, and nine pairs of graduated rods.

The latter, from a minimum length of 3 feet, increase 1.5 feet arithmetically to the maximum length, 15.0 feet. The graduations are tenths of a foot and are shown, on the top or upper foot of each rod only, by alternate rings of red and white paint; except the sixth ring, which is black. The top of each black ring is the immersion depth of its rod.

When made, each rod was separately tested and adjusted in a specially designed tank until, single or connected, its immersion depths differed only a few tenths of a foot: the depth was then painted on the rod.

By using a number of extension rods, float lengths of any depth can be obtained; but in practice the length is limited by the strength of the couplings: for the rods and couplings under consideration the limit is about 50 feet. The floats of round seasoned white pine, painted with three or more coats of white lead, are 2⅞ inches in diameter, in order to conform to the outer diameter of gas-pipe threaded sockets which are fitted on their lower ends. Each socket is loaded with sufficient lead to sink its float to the immersion depth, which is the top of the extension, and the black ring of the graduated rods, respectively. The extension rods have additional threaded sockets, flush with their tops, and sleeve couplings for making connections.

In order that the ends of the rods will abut, without the use of tools, all threads are thoroughly oiled and chased before use. To make the floats more conspicuous, while in use, a small hole is bored in the top of each graduated rod in which a small parti-colored flag is stuck: and to facilitate recovery a small staple is driven, above the immersion depth, to which a cord with surface float is attached. (Fig. 2.)

INSTRUMENTS.

Two transits, a stop-watch beating 0.2 seconds, and a sextant are the only instruments needed for a low water gaging; for medium and higher stages an additional transit is required.

SEQUENCE OF OPERATIONS.

The rangeman and a flagman set-up and orient a transit over each timing range-hub while soundings, from shore to shore, are being taken between the ranges. From these soundings float lengths ranging between maximum and minimum depths, are made up in pairs (precautionary for loss or breakage) and placed in order on the flat with signal flags and surface floats attached. The local

gage is read; the buoy skiff is anchored about 100 feet from shore and the observation skiff takes a position near by. To enable the floats to acquire the full current velocity an indeterminate buoy range, 80 or more feet above timing-range No. 1 is used by the buoy skiff. The tug moves slowly upstream while soundings are taken over the probable course of the float: by the time timing-range No. 1 is reached the probable float length is known and when the buoy skiff is reached the float is ready for launching.

Just before launching the float a flag is waved, as a signal to the range-man, and the tug stops; but immediately begins to slowly back while the float, bottom end downstream, is gradually lowered to the immersion mark and released, when vertical.

When the float approaches timing-range No. 1, the range-man depresses or elevates the telescope to the proper field of view; and starting the stop-watch, when float and range coincide, proceeds to timing-range No. 2. At timing-range No. 2 the observations of timing-range No. 1 are duplicated; the watch being stopped at the instant range and float intersect.

In the meantime, the observation skiff has followed the float at a distance of 3 to 6 feet: the flagman, on shore, signaling the skiff's approach to and crossing of the ranges. On signal that the skiff is crossing time-range No. 1 the observer, from the observation skiff, reads angle—timing-range station No. 1, downstream base end—and the leadsman, with drifting and standing lead, begins sounding; at conjunctivity of timing-range No. 2 and the skiff the observer reads angle—timing-range station No. 2, downstream base end—and soundings cease. The observation skiff now returns to the buoy range, selects float station No. 2, over which the buoy skiff anchors, and stands by. Meanwhile, the tug recovers the float and proceeds to sound over the probable float course as at the previous station.

At each succeeding station the same program is followed; care being taken that a satisfactory run is obtained. Finally, the local gage is read and the distance from the water edge to Arsenal and from water edge to timing-range station No. 1 is measured: in case of rapid oscillation a measurement is also taken at the beginning of operations.

To obtain the mean depth between the first and the last float stations and each shore, the observation and sounding skiff con-

tinues taking soundings and observations, in the usual way, between these stations and the water edge.

REMARKS.

In low water observations with consequent reduced current velocities only one range-man is necessary. At higher stages the flagman

Fig. 2.

is dispensed with, a range-man with a flag being placed at each range station; as the float approaches and crosses each range a flag is raised and dipped indicatively, likewise for the observation skiff. During high water gagings, time is kept by an assistant, on the floating plant, who starts and stops the watch at the dip of the flags.

At the higher stages the swift current precludes the use of a buoy

skiff and either prevents or greatly retards the return of the observation skiff to the timing ranges. When this condition obtains both the float and the observation skiff are picked-up by the tug, at the end of each run, and towed to the subsequent float stations which are located, with reference to shore objects, by eye. The writer made a gaging, at the southern city limits, in April, 1912 (St. Louis gage 30.7 feet), where the observed rod velocity ranged between 7 and 12 second-feet; as the usual tug could barely stem the current a tow-boat was used with flat lashed alongside. Previously, on account of danger from drift to the observation skiff, attempts were made to anchor the flat at float stations; but after three 150-pound anchors, placed tandem, failed to hold, these efforts were abandoned. At this gaging three ranges, 100 feet apart, were used and the floats timed at each; to prevent breaking at the joints, the longer floats (longest 46 feet) were coupled only when needed and towed behind the flat until a change of length was required.

Float stations have been charted and their sextant angles, with reference to base line, predetermined; but in practice, maneuvering the boat into position is both time consuming and difficult of execution.

Satisfactory gagings have been made with skiffs, during low water periods, the rods being transported, coupled and uncoupled in one of the skiffs; but the time required is much greater than by the method previously described. At the gaging of December 30, 1912, heretofore outlined, twenty-six one-float stations were used; the time required, including three reoccupied stations, was three hours. Very little oscillation of the float occurs if it is handled and launched as heretofore described; especially if the launching is done near the bow of the flat, thus allowing the float to immediately swing clear. Moreover, if a float strikes an obstruction or drags bottom its action immediately reveals the fact: operations are at once suspended and the flotation repeated with a shorter rod. During a continuous and extended series of gagings, both for comparisons and convenience, it is desirable to use the same float stations throughout. Buoys attached to wire cables having disc anchors jetted 15 to 20 feet into the river bottom have been tried; but drift, in conjunction with a small rise, will invariably tear them loose, not to mention the damage done by river craft. Since the floats project only about 6 inches above water, the effects of winds and waves are reduced to a negligible quantity; violent storms would

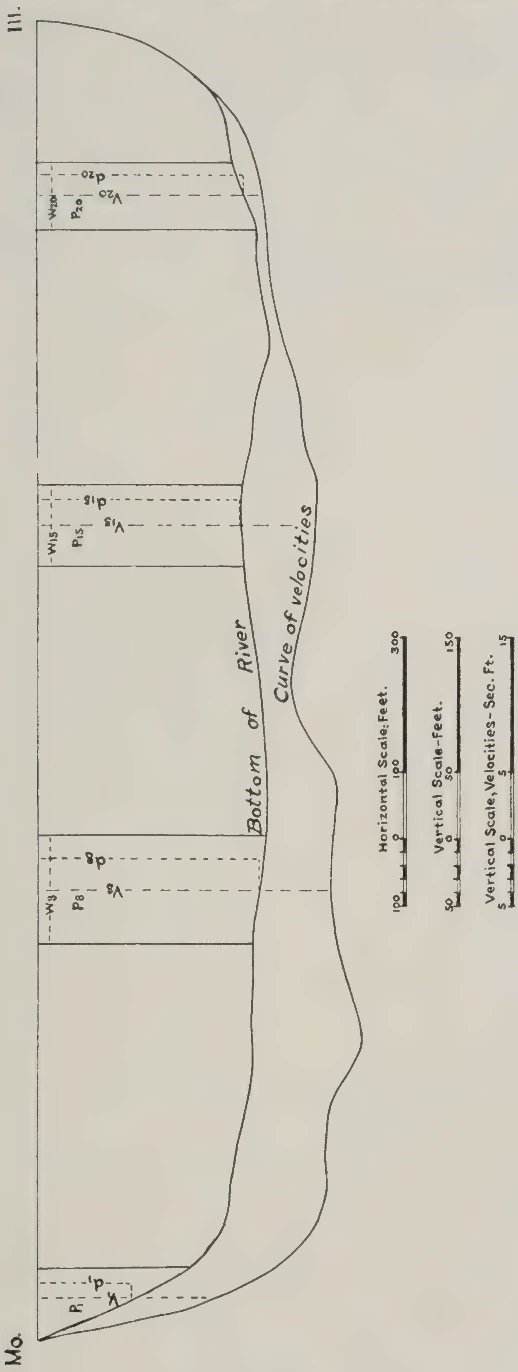

Ill.
Mo.
Bottom of River
Curve of velocities
Horizontal Scale; Feet.
100 0 100 300
Vertical Scale-Feet.
50 0 50 150
Vertical Scale, Velocities-Sec. Ft.
5 0 5 15

Fig. 3.

undoubtedly have a very appreciable effect on both the float direction and the velocity.

The determination of the float positions at timing-range No. 2 would be unnecessary were it assumed that constantly changing conditions of bottom, banks and stream flow would not materially deflect the course of the floats from normals to the ranges. As great changes in the sectional area frequently occur in short periods of time, accompanied by simultaneous changes of velocity, the above assumption would always be in doubt at any stage of the river; the uncertainty increasing as the rapidity of the oscillation increased.

When deviation of the float courses from normal to range lines is detected, operations are suspended until the base line can be swung and the ranges readjusted. Although preferable, it is not necessary that the rods float very close to the bottom of the stream, provided variations of data wide enough to reduce Vr to Vm can be obtained. Francis value of D was small, having been determined by floating rods in artificial rectangular channels or flumes; thereby securing a value under conditions that do not exist in natural channels. Moreover, Francis conditions the float length to, at least, nine-tenths the stream depth. The report of the Chief of Engineers, U. S. A., for 1883, page 2227, says of Francis' formula "the limits of the applicability of the formula were experimentally tested by comparing rods of different immersion with mid-depth floats and meters. The formula was found to give good results with an immersion of only one-third length." Table No. 1, compiled from the report of the Chief of Engineers, U. S. A., for 1901, pages 2209 to 2211, inclusive, shows the result of a set of comparative gagings—meters and rod floats—made at the U. S. Engineer Depot, St. Louis, Mo., in 1900. Computations: Gaging December 30, 1912.

The mean of the soundings over each float course between ranges was found; plotting these means as ordinates and the tangents of the observed transit angles (X 1000 for range No. 1 and 800 for range No. 2) as abscissæ, points on the cross section were located. When, as usual, sextant readings are taken, the cotangents of the observed angles are found and the abscissæ computed as before. On the ordinate through each of the above points the observed velocity Vr of the rod for that location, was plotted; a smoothly curving line drawn from water edge, through each set of

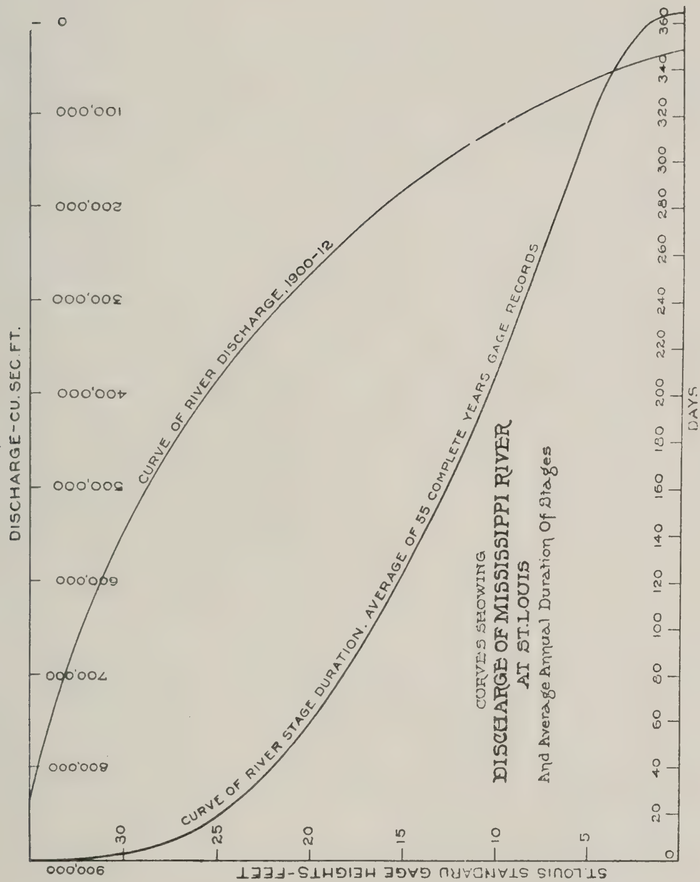

DISCHARGE-CU. SEC. FT.
0
100,000
200,000
300,000
400,000
500,000
600,000
700,000
800,000
900,000
ST. LOUIS STANDARD GAGE HEIGHTS-FEET
30
25
20
15
10
5
0
CURVE OF RIVER DISCHARGE, 1900-12
CURVE OF RIVER STAGE DURATION. AVERAGE OF 55 COMPLETE YEARS GAGE RECORDS
CURVES SHOWING
DISCHARGE OF MISSISSIPPI RIVER
AT ST. LOUIS
And Average Annual Duration Of Stages
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
DAYS

Fig. 4.

points, to water edge outlined the cross section and velocity curve, respectively.

The cross section was divided, by ordinates, into as many polygons p_1; p_2; p_3; —— p_{28}; as the sectional configuration called for. The widths w_1 ; w_2 ; w_3 ; —— w_{28} ; the mean depths d_1 ; d_2 ; d_3 ;—— d_{28} ; and the mean velocities v_1 ; v_2 ; v_3 ; —— v_{28} ; of the polygons were scaled (fig. 3): but $w_1 + w_2 + w_3 +$ —— $w_{28} = 1956 = W =$ in any gaging the computed width of waterway. From the above data the areas $a_1 = d_1\, w_1$; $a_2 = d_2\, w_2$; $a_3 = d_3\, w_3$; —— $a_{28} = d_{28}.\, w_{28}$; of the polygons p_1 ; p_2 ; p_3 ; —— p_{28} ; were computed. Then $\Sigma a_1 + a_2 + a_3 +$ —— $a_{28.\overline{\chi}} = 28753 = A =$ total area of cross section and $\Sigma a_1\, v_1 + a_2\, v_2 + a_3\, v_3 +$ —— $a_{28}\, v_{28.\overline{\chi}} = 55886 = V =$ total volume of discharge. Hence $V / A =$ the mean velocity and $A / W =$ the mean depth. The field record and subsequent computation for one float station is given in Table No. 2, while Table No. 3 gives the result of all observations, special and otherwise.

As previously noted, transit observations were a special feature of this gaging, hence the results in Table No. 3 derived from these determinations are superfluous, except for comparative purposes. The position of the floats, on the range lines, proves that the general direction of stream flow, float station No. 1 to No. 18 inclusive, was towards the left bank; while from station No. 19 to No. 26 the direction of flow shifted towards the right bank. However, not enough deflection, in either direction, was shown to warrant swinging the system and adjusting the ranges.

Table No. 4, compiled from reports of the Chief of Engineers, U. S. A., is a comparison of low water gagings, of approximately the same stage, at different periods. All of the gagings, included in the table, were taken at the regular station, U. S. Engineer Depot, except item 3, which was taken 3/8 mile farther downstream. This item gives the lowest discharge on record for this vicinity; but the result is considered doubtful. The first and second items are included because the river stages are records for gagings at this place.

However, any deductions made from Table No. 4 will have to take into consideration the effects of the contractive works (still efficacious) made in the fall of 1904 above the gaging station. This contraction caused a bar to form along the left bank, at the ranges, with deepening of the channel above and below the gaging station. Nevertheless, after making due allowance for disturbance of regi-

men, this table corroborates other evidence that a gradual lowering of mean bottom with simultaneous increase of mean depth is taking place in this vicinity, if not throughout this district.

The original discharge and duration of stage curves, "House Document No. 50, 61st Congress," have been slightly changed, due to the incorporation of additional data. (Fig. 4.)

CONCLUSION.

While most authorities favor current meters rather than rod floats, nevertheless rod floats properly manipulated give results that agree fully as well as those of the meter. The weakest point in rod float observations is the correction applied to the observed rod velocities—usually Francis' formula. This formula having been developed from observations taken under conditions not obtaining in natural channels and streams, is justly questioned as to its applicability. However, comparative observations both between rod floats and floats and meters indicate that the formula is not very far wrong. With an improved formula the writer believes that results more nearly approximating the truth can be obtained with rod floats than with meters, especially in streams with vagaries of current like the Mississippi.

The rod velocity is the sum of the movements of the stream filaments which act on it, hence boils, whirls, cross currents and eddies, as well as direct flow, are blended into the composite movement that governs the direction and velocity of the rod.

Portability, ease of manipulation, and computation are large assets in favor of the meter, but the .6 depth rule, for single meters, is arbitrary and good results therefrom are questionable in streams where boils, whirls and cross currents abound.

Either integration or simultaneous observations with several meters at varying depths are methods more likely to approach the truth. Nevertheless, the method of rod floats is believed to be based on correct principles, and will give better results, for streams with counter currents and shifting stream flow, than any other method.

Table No. 1.

Date of gaging, U. S. E. Depot.	St. Louis gage.*	Discharge cubic second-feet.		
		Rod Floats.	Current meters—	
			No. 29	No. 34
November 28, 1900	11.6	141971	144383	142005
December 1, 1900	10.0	120886	126425	123527
December 4, 1900 A. M.	8.6	103357	109683	109300
December 4, 1900 P. M.	8.6	103744	111594	109748
December 8, 1900 A. M.	7.4	89787	100114	102094
December 8, 1900 P. M.	7.4	92960	98960	97886
December 11, 1900	6.9	88110	92313	92080
December 14, 1900 A. M.	6.2	81841	84910	83505
December 14, 1900 P. M.	6.2	80378	81645	82291
December 19, 1900	4.8	66532	71674	71554
December 21, 1900	4.4	62243	69769	68880
December 27, 1900	3.8	60144	62099	62072
January 7, 1901	0.6	39891	41706	40399
January 10, 1901	2.2	49771	53553	52330
January 12, 1901	4.5	70244	72556	71476
January 28, 1901	4.8	70014	72788	72594
February 16, 1901	2.6	52721	55746	59601
March 2, 1901	4.1	65911	66492	65838

*Zero St. Louis gage = 379.79 feet above mean gulf level.

Table No. 2.

Float Station No. 2. Soundings: Range No. 1 to Range No. 2.	Angles to float— Sextant. Range No. 1	Sextant. Range No. 2	Transit. Range No. 1	Transit. Range No. 2	Float length.	Time.	Wind.
	73°—05′	68°—45′	17°—17′	20°—23′	*Feet.* 12	2′—0″	N. E.

Feet.	
15.0	Remarks:
15.0	Vr=1.67 (Rod velocity for 200.0 feet=distance between ranges.)
15.0	Vr=1.61 (Observed rod velocity for 200.3 feet=actual distance floated.)
15.0	D =0.15
14.0	
15.0	
14.0	
14.5	
14.5	
14.0	
13.5	
14.5	
14.5	
14.0	
14.0	
14.5	
14.0	
13.5	
13.5	
14.0	
14.0	
13.5	
13.0	
23)326.5	
14.19	

Table No. 4.

Date of gaging.	St. Louis Gage.*	Width of waterway.	Area of Cross Section.	Discharge cubic sec. feet.	Velocity sec. feet.	Mean Depth.	Oscillation.
Dec. 16, 1910	—1.42	1827	21764	32527	1.50	11.9	R.
Dec. 17, 1910	—1.25	1827	20370	33608	1.65	11.2	R.
Dec. 9, 1897	—0.10	1218	16730	24217	1.45	13.7	F.
Feb. 5, 1900	0.00	1925	14300	34600	2.41	7.4	S.
Jan. 16, 1913	—0.04	1909	24016	39188	1.63	12.6	R.
Dec. 22, 1904	0.20	2372	20300	39000	1.93	8.6	F.
Dec. 31, 1910	0.20	1842	22974	42569	1.85	12.5	R.
Jan. 10, 1901	2.20	2354	20600	49771	2.41	8.8	R.
Dec. 23, 1903	2.30	2401	22117	51495	2.33	9.2	R.
Dec. 30, 1912	2.20	1956	28752	58885	1.94	14.7	S.
Feb. 16, 1901	2.60	2330	22200	53000	2.37	9.6	R.
Oct. 31, 1910	2.55	1925	27214	59590	2.19	14.1	F.
Dec. 27, 1900	3.80	2379	23900	60000	2.52	10.0	F.
Dec. 8, 1904	3.80	2415	28205	69000	2.45	11.7	F.
Aug. 16, 1910	3.87	1885	28412	70366	2.48	15.1	R.

*Zero St. Louis gage=379.79 feet above mean gulf level.

Table No. 3.

Float Sta. No.	Dist. Float to Base Line. Sextant. Range No. 1.	Sextant. Range No. 2.	Transit. Range No. 1.	Transit. Range No. 2.	Mean Dist. Transit.	Mean Depth.	Vr 200 feet.	Vr, Actual Dist.	Polygon No.	Width Polygon.	Mean Depth.	Area, square feet.	Mean Velocity, feet second.	Discharge, cubic second feet.	Remarks.
	Feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*				*Feet.*	*Feet.*				
1	243	239	245	234	239	14.39	1.53	1.53	1	106	5.8	615	0.86	528.72	St. Louis gage
2	304	311	311	296	303	14.19	1.67	1.61	2	40	12.1	484	1.86	900.24	
3	326	320	322	314	318	14.76	1.61	1.62	3	40	13.5	540	2.05	1107.00	8.00 a. m.=2.20 ft.
4	412	401	408	399	403	16.37	1.75	1.76	4	80	14.3	1144	2.11	2413.84	
5	482	438	458	428	443	16.41	1.59	1.60	5	80	15.0	1200	2.16	2592.00	4.00 p. m.=2.20 ft.
6	591	583	587	586	586	17.14	1.84	1.84	6	80	15.8	1264	2.30	2907.20	
7	652	626	659	638	648	16.07	1.96	1.97	7	160	16.0	2560	2.32	5939.20	
8	749	728	733	719	726	15.80	1.96	1.96	8	160	16.3	2608	2.18	5685.44	Right water edge
9	778	750	777	776	777	15.00	2.09	2.09	9	80	16.8	1344	2.20	2956.80	
10	900	881	888	870	879	15.12	2.02	2.03	10	80	16.7	1336	2.09	2792.24	to Arsenal=456 ft.
11	909	893	892	889	890	15.08	2.01	2.01	11	60	16.3	978	1.92	1877.76	
12	1013	1009	1010	1010	1010	15.93	1.93	1.93	12	40	16.1	644	1.87	1204.28	
13	1091	1084	1094	1082	1088	16.25	1.87	1.87	13	80	15.7	1256	1.94	2436.64	Left water edge to
14	1167	1163	1161	1164	1162	16.67	2.04	2.04	14	60	15.2	912	2.00	1824.00	
15	1198	1194	1213	1207	1210	16.66	2.22	2.22	15	120	15.0	1800	2.05	3690.00	Base Line=90 ft.
16	1329	1314	1312	1297	1304	16.80	2.10	2.10	16	60	15.4	924	2.02	1866.48	
17	1334	1336	1335	1334	1334	16.66	2.27	2.27	17	60	15.8	948	1.95	1848.60	
18	1456	1460	1459	1463	1461	16.10	2.17	2.17	18	60	16.3	978	1.93	1887.54	
19	1465	1480	1461	1474	1467	15.71	2.31	2.31	19	40	17.0	680	1.84	1251.20	
20	1578	1575	1597	1591	1594	15.84	2.36	2.36	20	120	16.7	2004	1.74	3486.96	
21	1625	1631	1607	1619	1613	16.01	2.46	2.46	21	40	16.2	648	1.69	1095.12	
22	1679	1683	1688	1696	1692	15.52	2.11	2.11	22	100	15.2	1520	1.75	2508.00	
23	1755	1767	1742	1751	1746	15.05	2.19	2.20	23	60	14.2	852	1.57	1337.64	
24	1794	1803	1802	1800	1801	14.46	2.13	2.13	24	20	14.1	282	1.47	414.54	
25	1880	1888	1880	1884	1882	13.79	2.01	2.01	25	40	13.1	524	1.34	702.16	
26	1890	1891	1888	1882	1885	13.59	2.05	2.06	26	40	10.6	424	1.07	453.68	
									27	40	6.6	264	0.66	174.24	
									28	10	2.0	20	0.21	4.16	
										1956		28753		55886	

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Compiled by Henry E. Haferkorn, Librarian, Engineer School.

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(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institution of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review, London.
(80) Journal, American Society of Engineering Contractors, N. Y.
(82) Journal, New England Water Works Association, Boston.
(83) National Waterways, Washington, D. C.

*Now combined under title: Concrete-Cement Age.

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ENGINEER DEPARTMENT AT LARGE

WASHINGTON BARRACKS, D. C.

VOLUMES I–V

(1909-1913)

Subject-Index

Author Index.

Errata, Professional Memoirs

Volumes I to V

Volume I, page 319—

Ninth line from bottom: for the figures 0.96 and 0.98 read their reciprocals. 1.04 and 1.02.

Volume II, page 98—

A letter from Capt. J. A. Woodruff, Corps of Engineers, received after his article, A Combined Position Sketch (see page 50), had been printed, requests that the following corrections be made: Page 53, 7th line, for east read west; 8th line, for west read east in both cases; 16th line, for eastern read western; 17th line, for west read east; page 54, 1st line, for western read eastern; 2nd line, for east read west.

Volume II, page 262—

Last line: page 167, read Janvier; not Janiver. Thirteenth line from bottom page 173, read 2 of 4, not 2 to 4.

Volume IV, page 324—

In the last part of the second term of the equation on page 591, the denominator should read 1 + and the parenthesis around the numerator should be omitted.

Page 596, line 15, the word "footing" should read "footings," and in line 17, the words "center of" should be stricken out.

Volume IV, page 668—

In connection with the article by Lieut. James G. Steese on "The Corps of Engineers and the Isthmian Canal," which appeared in the July-August number of the PROFESSIONAL MEMOIRS, the following correction should be made:

Col. Wm. M. Black (then Major) and Lieut. Mark Brooke, Corps of Engineers, were placed under the orders of the Isthmian Canal Commission on April 1, 1903, instead of March, 1904. They proceeded to Panama and were stationed at Culebra. The duty assigned to Colonel Black was to observe and report on the work of the new Panama Canal Company, reporting particularly on the expenditures made month by month and the work accomplished. Colonel Black was absent from the Isthmus May 4, 1904, when the actual transfer was made, but returned thereto a few days after and remained as Acting Chief Engineer of the Commission until the arrival of Mr. John F. Wallace, who took charge of the work, July 1, 1904.

Volume V, page 248—

The following table of costs is furnished by Maj. E. H. Schulz to replace table of cost shown on pages 710 and 713, PROFESSIONAL MEMOIRS, No. 18.

a. Willow Mattress.

757 cords brush, at $0.892	$675.87
776 cubic yards ballast, at $0.746	579.09
555 cubic yards spalls, at $0.702	389.70
Wire strand, clips	204.43
Towboat service	784.00
Labor, superintendence and miscellaneous	2,669.59
	$5,302.68

b. Concrete Paving.

1,156 cubic yards gravel, at $1.533 plus	$1,772.35
Grading bank, 1,000 linear feet, at $0.946007	946.07
Reinforcement	749.32
821¾ barrels Portland cement at $1.15	945.01
Labor, superintendence, and miscellaneous	1,881.33
	$6,294.08

c. Flexible Concrete Blocks.

120 cubic yards gravel, at $1.533	$183.96
Reinforcement	221.65
182 barrels Portland cement, at $1.15	209.30
Labor, etc., making blocks	636.00
Labor, etc., placing blocks	685.12
	$1,936.03

Summary.

Weaving and ballasting mattress, 1,030 linear ft., at $5.1482	$5,302.68
Concrete paving and grading, 1,000 linear ft., at $6.294	6,294.08
Flexible concrete blocks in place	1,936.03
Total	$13,532.79

Volume V, page 374—

The exponent of the fraction in top line, page 145, should be 2-3 instead of 3-2.

The word ''left'' which ends the first line of the caption to Fig. 1, page 33, should be ''right.''

The words ''Fig. 1,'' at the end of line 10, page 184, should read ''Fig. 4.''

On Plate 2, opposite page 250, near the lower left-hand corner, the time of maximum northbound tide should read ''the VII lunar hour'' instead of ''the XII lunar hour.''

It should be noted that the part of the curve in the middle of Plate B, page 253, below the line is the discharge for southbound currents and that above, the discharge for northbound currents.

In the formula, page 265, the second term of the equation, only, is divided by 4.

On page 278, 4th paragraph, 7th line, there should be a period after the word ''kills;'' the word ''under'' should be omitted; the word ''the'' begun with a capital; and the words ''was therefore made'' inserted immediately after the word ''assumption.''

Volume V, page 496—

Page 473, 19th line from the top, change the word ''not'' to ''now.'' Same page, 5th line from the bottom, replace the words ''it is'' by ''if.''

Page 477, 16th line from the top, add the word ''have'' after ''and.'' Same page, 20th line, add the word ''a'' after ''make.'' Same page, 23rd line, add the word ''out'' to ''lays.''

Page 479, 20th line from the top, change the word ''involved'' to ''involves.''

PROFESSIONAL MEMOIRS
Corps of Engineers, United States Army, and Engineer Department at Large

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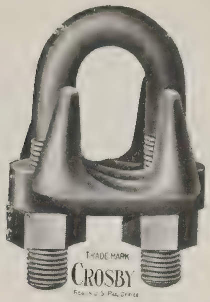
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